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Front Cover: CBA Banking Chamber, Melbourne: The original architectural drawing prepared in 1890 (Photo: Ian Mckenzie Photography Pty. Ltd.)

Back Cover: OCBC Building, Singapore under construction (Photo: Kouo Shang-wei.)

Foreword

John Nutt

When I was in London in September 1975 I suggested to Jack Zunz and Peter Hoggett that if we could discipline ourselves enough to write the articles, there were enough interesting projects being undertaken by the Australian Partnership to fill an issue of *The Arup Journal*. The alacrity with which this suggestion was seized and the commitment of a date of issue imposed this discipline and here is the result. It is the first time we have gone into print on these projects although some like OCBC have been the cover articles of *Engineering News-Record* and *New Civil Engineer*.

We have selected the projects that best give a balanced view of the work we do, which is a mixture of structural, civil and traffic. All our offices are represented.

Many of the people, both senior and junior, who have contributed to these projects in a significant way are not mentioned. It is the nature of writing an article that authorship falls on one or two individuals who can best report on the concept and development of a project and who can convey the experience and philosophy of the practice.

The nature of our practice is changing. Our reputation in Australia was initially founded on the design of buildings and we have worked with some of the best architects in Australia on commercial, institutional and governmental projects. With the changing economic climate, as severe in Australia as in other parts of the Western World, the building scene has suffered a critical restriction and it will be several years before it returns to the level of activity that has characterized the last few years. We have found that we can successfully export our services and this will become an increasingly important part of our work, particularly in the countries of South-East Asia.

In civil engineering, working alone or in multidisciplinary teams, we will be able to make an Australian contribution to this country and its neighbours. When we come to write our next Journal – if we can ever discipline ourselves a second time – the contents will probably be significantly different.

Introduction

Jack Zunz

The Opera House has become a familiar and famous landmark in Sydney's harbour. It has contributed immeasurably to Australia's cultural life and its design and construction are well recorded in the annals of modern architecture.

In writing about the Opera House much has been said about its planning, design and construction as well as its financial problems and the political consequences of delays and difficulties. Not much has been said about the personal stories, both comic and tragic, of the people concerned with the creation of such a controversial and technically demanding project. Nothing at all has been recorded about what, for Arups, has possibly been the most important consequence of its construction, namely the establishment of a permanent Arup Office in Australia.

This issue of the Journal aims to help fill the void and is devoted totally to the work of our Australian Partnership.

The Partnership emerged out of the group of some of our most talented engineers who went to Sydney to complete the design and supervise the construction of the Opera House. Paradoxically the very scale and publicity associated with the building meant a diminution of recognition for other equally demanding but less spectacular and newsworthy projects. One issue of *The Arup Journal* will not do justice to all the work being done and can only describe a limited number of projects.

The development of the Australian practice has taken place at a time of increasing national awareness and greater self-reliance on the part of the Australians. The influence of the United Kingdom in cultural, scientific and technological matters, once omnipotent, has been reduced to sensible bonds of kinship. Even America's powerful influence is not what it was and the country's relationship with its Asian neighbours, its geographical isolation and its virile and forward-looking people have resulted in a new-found independence and the emergence of a real Australian ethic.

In matters to do with the built environment, Australian architects and engineers have contributed much and their standards are matched by few and are the envy of many. Our office has played and is continuing to play its part not only in the country's exciting development, but also in helping the less privileged communities in newly independent Papua New Guinea, as well as in some Pacific Islands.

The achievements of what is one of our younger partnerships are formidable. From a block of home units (flats) in Bondi Beach to a spectacular 60-storey office building in Singapore, from a jetty in the Solomon Islands to the infrastructure of a township in Perth, the work is varied, interesting and challenging, and contributes to our collective experience and expertise. We can all take some reflected pride in these achievements.

In using abstractions like 'partnerships' or 'office' we obscure the personalities, the people who have actually made it happen. Our success in Australia has been based on a relatively young, dedicated, stable and strong leadership, or as Emerson said more succinctly, 'All history resolves itself very easily into the biography of a few stout and earnest persons'.

What more can I say?

The OCBC Centre Singapore

Peter Thompson

The OCBC Centre is a banking/office complex in the commercial heart of Singapore being developed by the Oversea-Chinese Banking Corporation as part of the Singapore Urban Redevelopment Scheme.

The Bank's head office has traditionally stood on part of the site and the previous structure, known as the China Building, was a well-known local landmark.

The new building, apart from becoming the Bank's head office, will provide some 50,000 m^2 of rentable office accommodation in a 51-storey tower plus parking for 660 cars in a separate six-storey parking structure. The eventual cost will be of the order of $,100 m. (Singapore) including land acquisition.

The accommodation is arranged as follows:

At ground level there is a banking chamber 30 m wide by 52 m deep, which rises through four storeys. Above this are three similar tiers consisting of 14 (35 m × 31 m) office floors above a 35 m × 20 m plant room. Above these three banks of floors are two further floors of 35 m × 20 m, one of which is a penthouse for the Bank Chairman, the other a plantroom.

Vertical transportation and services are housed in two 20 m diameter semicircular service cores which stand 35 m apart at opposite ends of the building and which give the building its distinctive vertical expression.

The building stands 198 m from the street level and has one basement.

The architect for the project is I. M. Pei of New York in association with BEP Akitek of Singapore.

Apart from normal high rise planning considerations, two factors greatly influenced both the architectural form and the construction method finally adopted.

Firstly, it was the Bank's wish that the banking hall should be as imposing as possible and completely unobstructed by columns.

Secondly, under the Urban Redevelopment Scheme, owners of projects which are completed in a given time after planning approval are granted significant taxation benefits. For this project the potential long-term saving could be in the order of $50 m. and the Bank was anxious that the design be such that it could be built quickly.

In April 1972 we were asked by the management consultants for the project to give an opinion on the structural concept for the building. We proposed the adoption of tiered construction in order to achieve savings in time and cost.

The column-free banking chamber could be achieved by supporting each bank of floors at its lower plant room level on girders which would span between the service cores. It logically followed that if the service cores and plant room girders could be constructed at an early stage, then the potential would exist for construction of more than one level at a time, thus speeding the building process.

Our report submitted in May was based on the service cores being slipformed independently ahead of the remaining structure in order that transfer girders could be erected at 20th and 35th plant room floor levels as soon as possible. These girders were envisaged as basically steel members. For a variety of technical reasons, the lower fourth floor level girder was envisaged as prestressed concrete supported on columns immediately adjacent to the service cores, and not obstructing the banking hall.

From these transfer girders would spring the three banks of typical floors on which work could proceed simultaneously. Our comparative time schedules showed a saving of one-third over conventional 'from the ground up' construction.

This report was accepted and we were appointed structural and hydraulic engineers, charged with producing all working drawings for a fixed price superstructure contract in the six months to January 1973. In addition, during this time, piling and foundation work was to be designed and constructed such that the superstructure contractor could commence work on site off prepared foundations in April 1973.

This would have been difficult given ideal circumstances, but with the design architects in New York, other consultants in Singapore and ourselves in Australia, the task appeared formidable.

Ron Bergin, who had worked on the original submission, was posted to Singapore to be resident engineer for the foundation works and, more importantly, to act as our liaison with the other consultants. The design work was carried out in our Perth, Western Australia office.

It must be stated at this stage that we were ably assisted by Ove Arup and Partners Singapore who, apart from being responsible for the total hydraulic design, produced the working drawings for the foundations and car parking structure.

Fig. 1
OCBC Centre
(Photo: Kouo Shang-Wei)

Fig. 2
China Building, previous head office of the Oversea-Chinese Banking Corporation
(Photo: Courtesy of OCBC)

(a) January 1975

(b) May 1975

(c) June 1975

(d) September 1975

(e) April 1976

Fig. 3 a, b, c, d, e
Progressive construction
(Photos: Kouo Shang-Wei)

The building was fairly readily broken down for design and construction into the following elements which will later be described:

(1) Foundations
(2) Service cores
(3) Typical floors
(4) Transfer girders and their associated plant room floors
(5) Upper level floors
(6) Parking building
(7) Cladding.

The Codes of Practice used in design were generally the relevant British codes with the proviso that if an Australian code was more severe this was to be used. For the steel girder fabrication, American Bureau of Shipping standards were applied at the request of the fabricator.

Piling, excavation and pile cap construction formed separate initial subcontracts let on the basis of provisional quantities in June 1972, and timed for completion in March/April 1973 for immediate takeover by the main superstructure contractor. A tender submitted by Low Keng Huat Constructions was accepted and the contract successfully completed on time.

It was considered that the superstructure contract would be outside the competence of all but the two largest local contractors, both in terms of size and technical complexity, and applications for tender registration were invited internationally, from which a short-list of 10 (two each from Singapore, Japan and America and one each from Australia, Germany, Hong Kong and Korea) was chosen.

Prior to the issue of tender documents preliminary drawings were issued to the contractors and two series of pre-tender familiarization meetings held with them. These were to make them fully aware of our intentions or aspirations and, almost as importantly, for us to receive feedback which could possibly result in a more meaningful set of documents.

As could be expected, not all the contractors shared our view that tiered construction would be beneficial and it was subsequently agreed that tenders would be invited on the basis of floor-by-floor as well as tiered construction; the main difference, apart from procedure, being that prestressed beams could be used in lieu of the steel girders at 20th and 35th floor levels.

Of the 10 contractors, two did not tender and the contract on a tiered construction basis was awarded to a consortium of Morrison-Knudsen of the USA and Low Keng Huat, the local contractor for Stage 1. Their programmed time for completion was 926 days compared with the shortest time of 1,286 days submitted for conventional construction. Although the latter was $6 m. cheaper, the time difference was more valuable to the owner.

Foundations

Some 12 boreholes were drilled on the site to a maximum depth of 85 m and soils existing on the site can be shown to fall into four main categories:

(1) Fill overlying loose sand to an average depth of 5 m

(2) Soft marine clay to an average depth of 5m

(3) Loose sand to 23 m

(4) Stiff to very stiff multi-coloured sandy or silty clays. This formation becomes harder at depth with the inclusion of sandstone.

As with most structures built in this area of Singapore, the obvious answer was to found the building on steel piles driven to a set in the stiff clay layer.

Pile tests were conducted using 356 mm× 368 mm steel piles driven with various hammers to prove that penetration of 14 m into the hard layer was achieved with a set of 10 mm in 30 blows. These piles were subsequently tested satisfactorily to a load of 4000 kN and a working load of 2000 tonnes was assumed.

Extraction tests were carried out and it was shown quite clearly that the piles were being gripped firmly over the full depth of the hard layer. The piles under the tower block are spaced generally on a 1.2 m× 1.2 m grid. Massive pile caps 5.5 m thick, each with 280 piles, support the service cores and columns to the fourth floor girders, and they are connected by a 1 m thick slab which forms a security floor for the vaults and strong rooms situated in the basement.

Piles to the car park structure are of similar section, but driven to a lesser set for a working load of 1700 kN. Larsen IV sheet piles are supplied around the tower to act as a coffer dam and retaining wall during excavation to a maximum depth of 8.5 m.

The excavation was planned in two stages, the first to the underside of the 1 m slab in the centre of the building. This was cast and the sheet piling propped from it using 700 mm diameter steel supports from the slab to walers around the perimeter.

Where the excavation was surcharged by adjacent buildings, flat jacks were incorporated for pre-loading prior to Stage II excavation.

The excavation was continued down to the underside of the main pile cap using clam type drag lines operating from two bridges constructed across the site. Due, however, to the closeness of the grid, much excavation was done by hand.

The excavation was not completely watertight, and dewatering was necessary at some locations. Unfortunately, one of these was next to

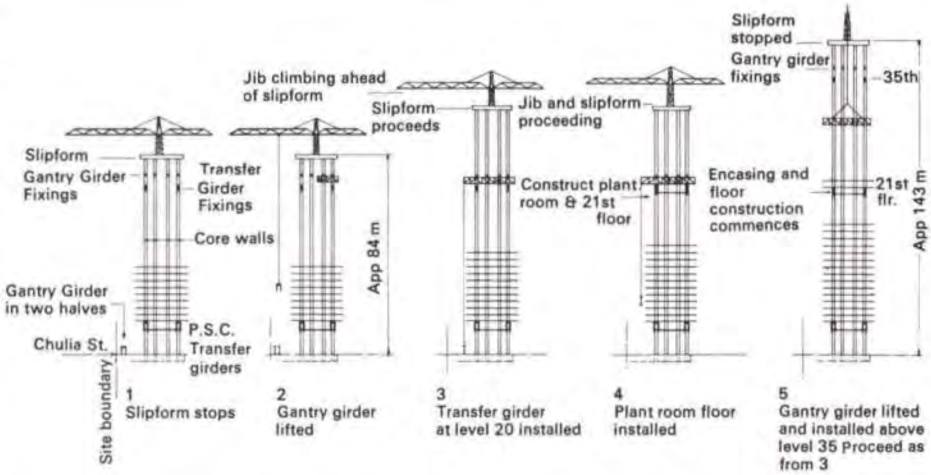


Fig. 4
Erection sequence as proposed

Fig. 5
Foundation piling
(Photo: Kouo Shang-Wei)

Fig. 6
Basement excavation
(Photo: Kouo Shang-Wei)

some adjacent buildings which after a while started to sink slowly into the mud. We tried to stop this by pumping cement grout under the building but this was ineffective. However, we just managed to complete our concreting and stop dewatering before the injunctions arrived and the resulting repairs probably improved the properties which were in bad order.

Service cores

The service cores are the main vertical supporting elements carrying gravity loads from the second tier of floors upward and all lateral loads from wind only, seismic loading not being considered.

Apart from minor components of loads from the floors and self-weight, loads are applied at girder positions. Our design problems were simplified by the inclusion of bearings at these points, giving us a semi-articulated structure and avoiding secondary stresses being set up due to prestress, creep, shrinkage, temperature, etc., of the girders.

Moments in the core due to eccentricity of girder reaction were taken out by push-pull action of the floor slabs. As the load of the lowest tier of floors did not contribute to this moment, eccentricity of load at pile cap level was nominal only. Total vertical loading at the base was 400 MN.

We had only limited data as to design parameters for wind loading in Singapore, where a 10-second gust at 30 m/s was the normal design criterion for high buildings. After consideration we used a basic wind speed of 32 m/s corresponding to a three-second gust at a return period of 50 years and assumed Terrain Category 2. We also carried out an investigation into the dynamic response of the structure, this being a particular requirement of the clients.

We commissioned the University of Sydney to carry out an aerodynamic investigation of the building, and their report showed that we had been fairly conservative in our estimation of wind speed, 22 m/s being recommended as adequate after extrapolating from results obtained over a limited number of years at two separate locations in Singapore.

However, this investigation could not be completed until February 1973 so that their report, which, apart from the aspects of design wind speed, included dynamic response and environmental effects, came too late for us to change our design appreciably.

The cores require a vertical load precompression to counteract tensile stresses due to wind. The bottom 19 floors do not contribute to the compression but to the wind forces, and the relative progress of core and floor construction was monitored throughout to ensure that a potentially dangerous situation would not arise.

The cores were designed for slipform construction with form changes kept to a minimum and generally at girder locations. Reinforcement which was a maximum of 1.7% at basement level, was arranged in separate cages between yoke positions which were prefabricated and installed using the tower cranes.

Concrete strength varied from 38 kN/mm^2 to 31 kN/mm^2, and the walls were slipformed in 3.66 m lifts containing 183 m^3 of concrete at the lowest levels. Concrete in walls was poured in one day with an overall turnaround time which varied from four to five days. Ready-mixed concrete was used as for all other parts of the job and a retarder was incorporated as a safeguard against delays in delivery caused by traffic congestion.

Verticality and twist were checked by plumb line which, in these days of lasers, etc., still seems to give the most accurate results. A survey was carried out at five levels by an independent firm to check the contractor's results. These results rarely tallied with the contractor's, but the differences were small enough to be overlooked.

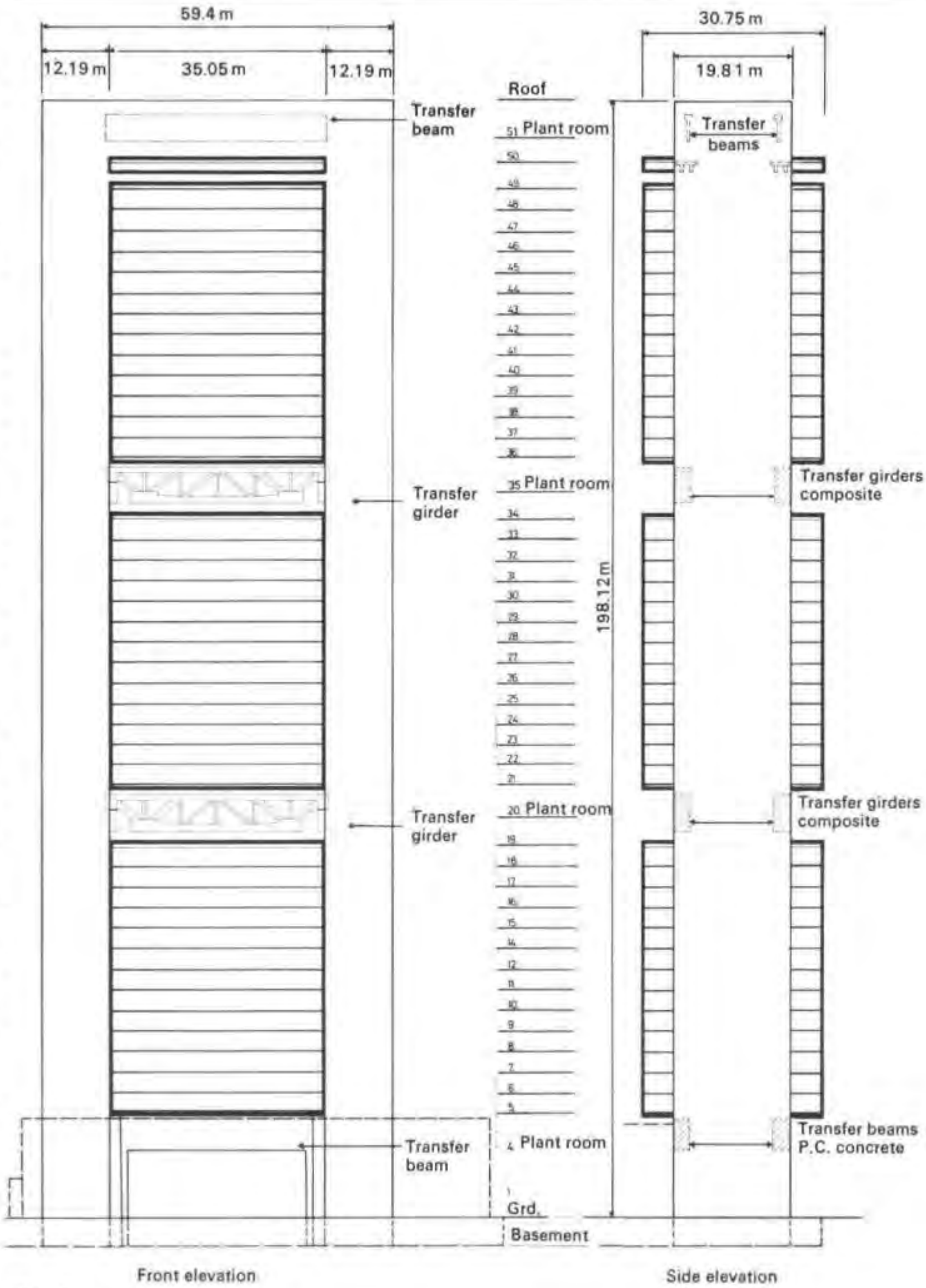


Fig. 7
Front and side elevations

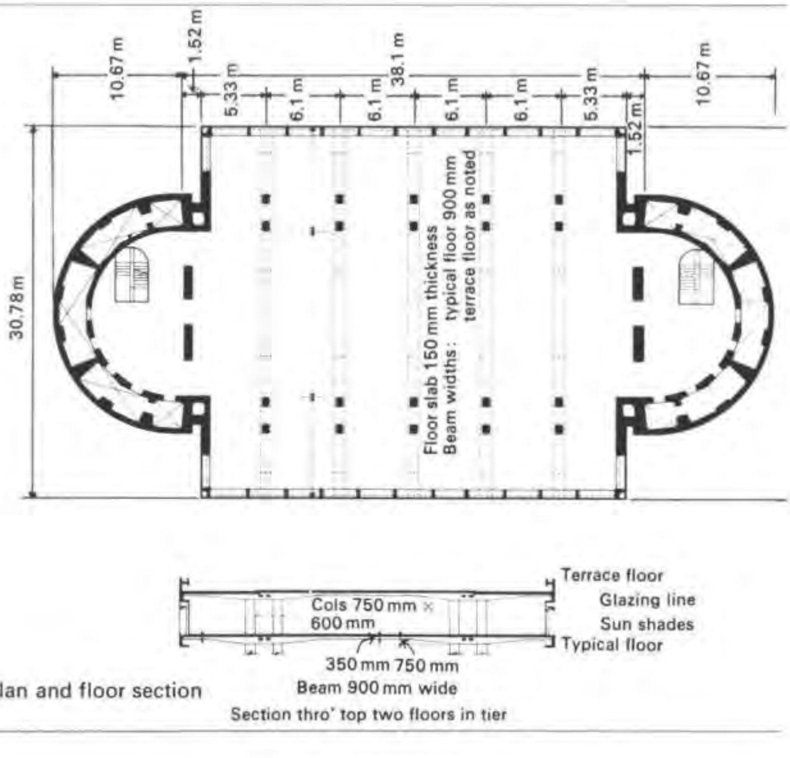


Fig. 8
Typical floor plan and floor section

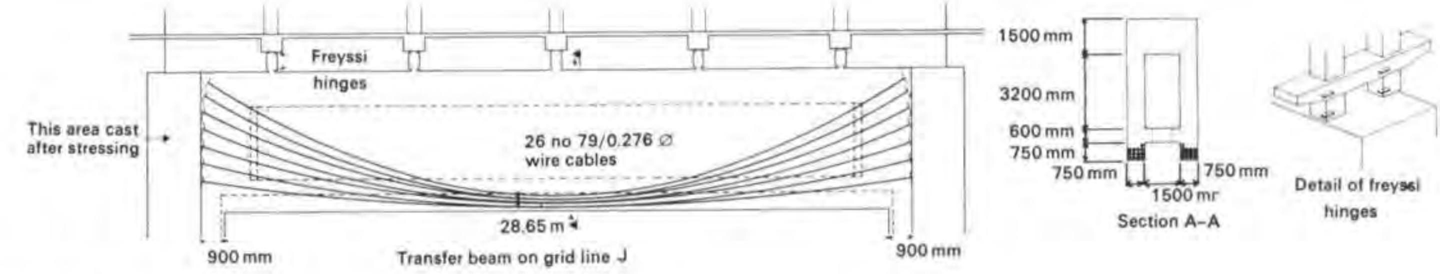


Fig. 9
Prestressed beam and section at Level 4. Detail shows floor columns above and Freyssinet hinges

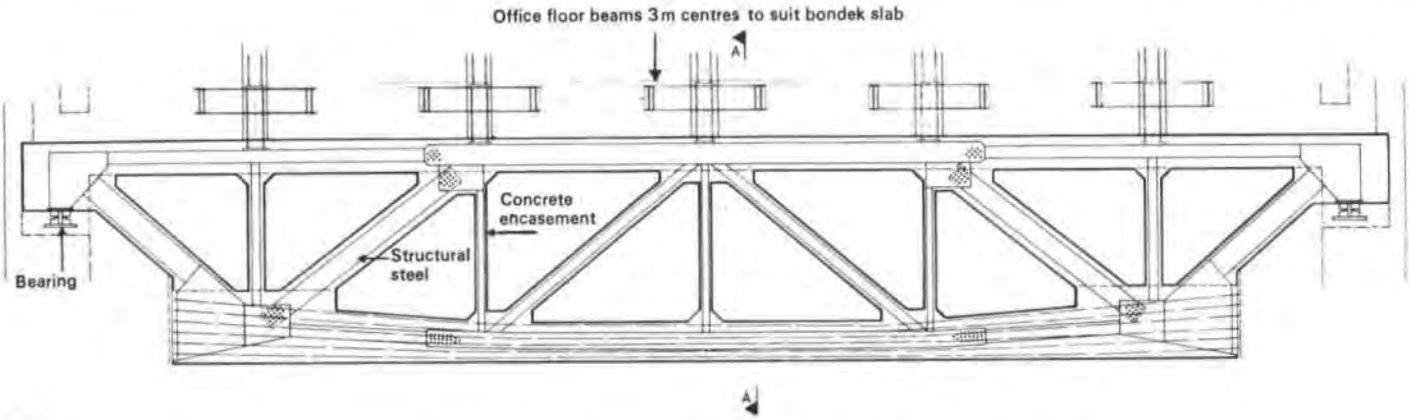


Fig. 10
Elevation of transfer girders at Levels 20 and 35 showing structural steel, prestressing cables and encasement

The slipform subcontractor was Fabquip Ltd., an Australian firm with an almost 100% monopoly in Australasia. The system is an all-metal one with a high degree of geometrical control. The walls were consequently within the vertical tolerances laid down of 6 mm in 3.66 m and 25 mm total.

Prefabricated blockouts were left for the core floor slabs and beams which followed on about five levels below the slipforms.

Concrete cube tests were made at seven and 28 day intervals for the core wall concrete and, at two levels, extensive core testing was carried out to prove the adequacy of the concrete after low results had been obtained. Although the concrete was subsequently approved, the procedure of testing, etc., was time-consuming and a decision had to be taken to stop the job or allow the contractor to proceed at his own risk. Either of these decisions can bring unwarranted pressure to bear on the engineer and this case was no exception. However, the eventual advent of wet testing will do much to eliminate this problem.

Typical floors

The main floors are 35 m× 31 m wide with 5.5 m projections each side of the line of the service cores. The upper floor in each tier has an open terrace over the area projecting from the core. The loadings were standard office with lightweight partitions and 50 mm screed. Columns were permissible within a 3 m wide strip from the line of the service cores corresponding with the transfer girders below, so that a large cantilever was inevitable. A concrete sun screen system was incorporated adding to the load at this point.

As we had 42 more or less typical floors, we investigated a variety of floor systems in reinforced and prestressed concrete and on purely economic grounds chose a conventional beam and slab system with a 150 mm slab spanning 6 m and 900 mm wide beams which vary in depth from 350 mm at the edge of the building and on the centreline, to 760 mm at the columns. These beams span over twin columns 900 mm× 550 mm placed on the edge of the 3 m strip.

The columns were maintained at this dimension throughout the height of the tier to minimize deflections of the cantilevers, this being particularly important for the uppermost level which was column-free and had increased cantilever loading from an upstand parapet/flower box. The calculated long-term deflections ranged from 65 mm to 100 mm and cambers were incorporated to compensate.

Transfer girder at fourth floor level

As has been said previously, an early decision was taken to construct this girder in prestressed concrete on columns independent of the core. This decision had three significant effects. The span of the beam was reduced from 35 m to 30 m, the distribution of load on the foundations was improved and the slipform could pass unhindered.

The girders are loaded from above by their tier of 14 floors and on their bottom edge by the plant room and the banking hall roof (one side only), the maximum load being 70 MN after allowing for a 50% reduction in live load from the upper floors. They form the external wall to the plant room and are cast in situ box sections 6 m deep× 3 m wide, with web thickness of 760 mm and top flange of 1.5 m. They are solid sections for a distance of 3.6 m adjacent to the anchorages.

We were limited to a concrete strength of 38 N/mm^2 which led to rather thick sections. The prestressing system used was BBRV with local (Singaporian) modifications and cables consist of 7 mm diameter wires with ultimate tensile strength of 1760 N/mm^2. Each cable is stressed and shimmed off at 70% of its ultimate capacity and relaxation of this stress level was specified as 5% at 1000 hrs. We subsequently used these cables as standard for all prestressing throughout the job.

The front girder was prestressed using 26 cables alternately stressed from one end only. Cables were stressed in two stages, 10 seven days after completion of casting and the rest after nine floors had been built above.

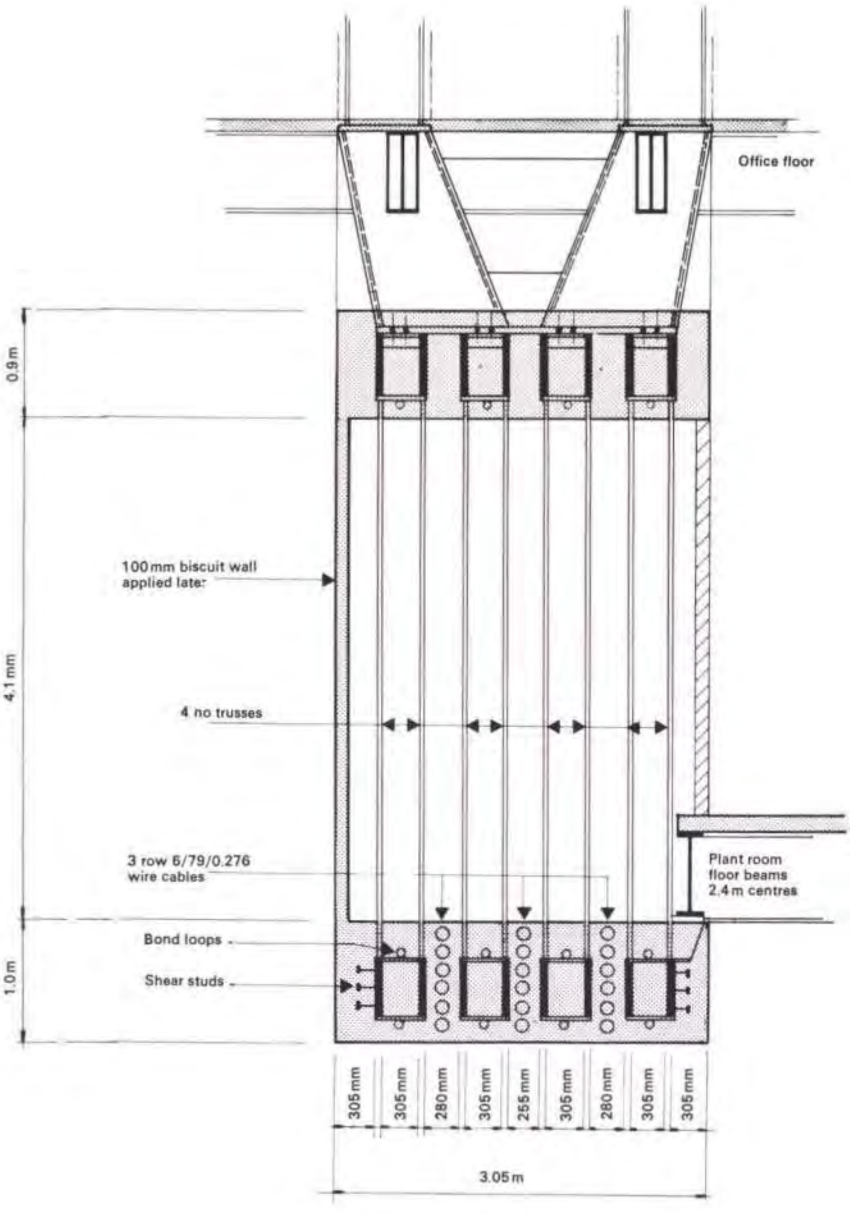
The plant room at this level housed most of the large items of equipment such as chillers and we were required to provide an opening through the beam adjacent to the banking hall roof to allow access of equipment. This resulted in a notch 4.27 m long× 2 m deep in the top of the beam in the most advantageous position to ourselves which was 3 m from the support.

Even in this 'advantageous' position the prestress force required rose by 40% to 34 cables, stressed in three stages and gave us a difficult stress distribution problem around the notch. We carried out a rigorous analysis of this area using finite element methods and a a result incorporated 1100 kN of vertical prestress per foot run of beam over a 6 m long zone to reduce the principal tensions.

For this we used 32 mm diameter prestressing bars stressed to 50% ultimate capacity. We argued against incorporation of this notch for obvious engineering reasons but were overruled. You can imagine our feelings when we eventually discovered it was unnecessary.

To ensure that the girders were able to deform as the cable forces were applied, releases were left in the structure and Freyssinet type hinges were incorporated at top and bottom of the short stub columns which transmit load to the girder from the slabs above. The supporting columns, although 3 m× 1 m in section, stood 10.5 m above their nearest horizontal restraint and were very flexible in the direction of prestress force. The releases which were slots left in the plant room floor were cast up after final stressing and the columns subsequently attached to the core to form a riser duct. Although not theoretically necessary, the hinge throats were dry packed some time after movement had ceased.

The girders were cast in four stages to construction joints specified on our drawings. The concrete work was not ideal and some problems arose. On stripping the formwork adjacent to the anchorage zone we were confronted with voids bigger than footballs behind the plates. As we were applying a force of 4000 kN



Section A–A

Fig. 11
Cross-section through transfer girder

at transfer we were naturally worried but, after extensive cutting out and replacing, the concrete proved adequate. About 10% of the prestressing ducts had suffered quite a lot of damage due to the concrete dropping a considerable distance onto cable sheaths which were too thin. Ingress of concrete caused blockages and it was some time before we could free the cables sufficiently to obtain satisfactory extensions. Grouting was also impossible and extensive drilling had to be carried out to locate the cables and provide access for grout tubes.

On stripping the bottom forms we found some cables completely exposed with the ducts broken and corrosion of the stressed cable was very evident. A plastercast using rapid hardening cement was applied to the beam to enable cable grouting which was eventually removed and the area made good.

Banking hall

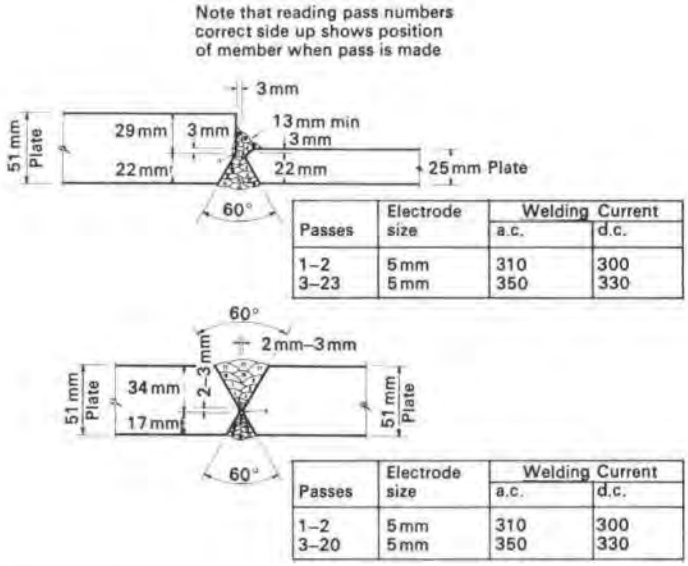
The soffit of the transfer girders and the plant room between them forms part of the ceiling to the banking hall. The remainder, an area of 30 m× 30 m, is supported on three sides by walls and on the fourth by the transfer girder. This area is designed to carry car park loading but has an electrical sub-station over a small area.

The structure, which could loosely be described as architectural, consists generally of 2.1 m× 600 mm wide beams at 3 m centres in each direction with a 100 mm slab spanning between the beams. This appears fairly massive but the coffers encase the hall lighting and openings of up to 60% of the vertical area were required through the beams to permit circulation of cooling air, making a relatively easy problem rather more difficult to solve. The area was analyzed as a grid using modified torsional and stiffness coefficients and designed in reinforced concrete apart from a small area under the sub-station where prestressing was used.

The banking hall proper has a mezzanine floor 3.6 m above ground floor level which appears to cantilever from the perimeter walls, but which is supported on 30 m span prestressed beams. Access to this floor is via two self-supporting circular stairs.

In all, the impression is one of vastness and sufficiently intimidating to the average customer to inhibit requests for overdrafts!

Fig. 12
Erection of transfer girders at Level 35
(Photo: Kouo Shang-Wei)



Passes	Electrode size	Welding Current a.c.	Welding Current d.c.
1–2	5mm	310	300
3–23	5mm	350	330

Passes	Electrode size	Welding Current a.c.	Welding Current d.c.
1–2	5mm	310	300
3–20	5mm	350	330

Fig. 13
Transfer girders: plate welding sequence

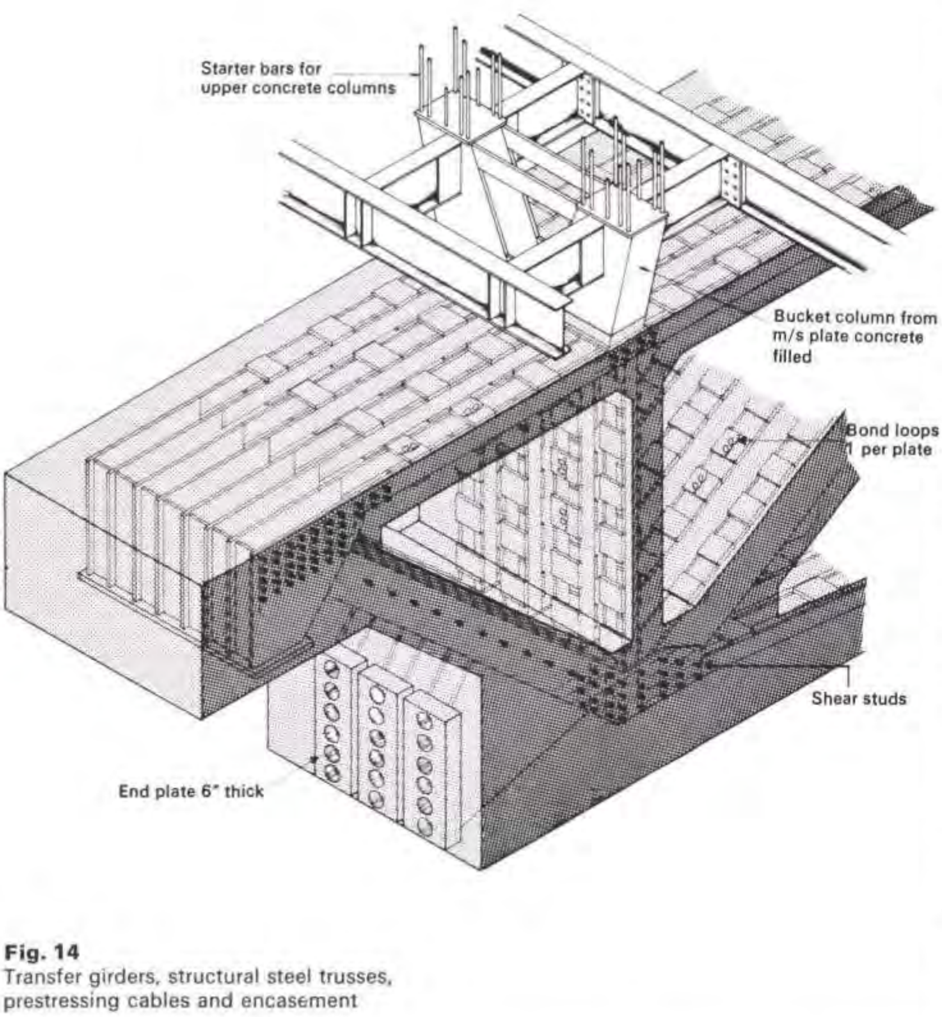


Fig. 14
Transfer girders, structural steel trusses, prestressing cables and encasement

Upper transfer girders

From the outset it was realized that the only practical way of initially bridging the 35 m gap between the service cores, firstly 76 m then 137 m above ground, was by using some form of steel construction which would become part of the permanent works.

We had considered prestressed concrete beams either in precast segments or poured in situ, but the temporary staging necessary was clearly economically prohibitive for a building structure.

The use of tower cranes for lifting this steel structure was ruled out as it would either mean bigger cranes than could be profitably employed on the remainder of the normal work, or pieces of steel so small as to maximize rather than minimize the amount of work to be done in the air.

A survey of the winching equipment available showed that a lift of around 50 tonnes should be considered as a maximum. As at fourth floor level we had an envelope of 6 m × 3 m for the girders and, by normal process of design evolution, established that we would place within this envelope four or five steel trusses with sufficient space between them for prestressing cables with the necessary attachments for ensuring composite action when they were eventually combined by encasement into one very large truss.

We thus had 200 – 250 tonnes per girder of high tensile steel available in the form of steel trusses on which to base our progressive strength design.

We assumed that compression members would act compositely after encasement and that with prestressing, the bottom tensile chord would act compositely, thus minimizing the amount of steel required. All other tension members were assumed to be steel only.

The girders were designed as rigid frames on pinned supports loaded by 71,000 kN at the node points. In addition to the more normal design criteria an important objective was to ensure that the constituent parts of the composite members approached their maximum working stress levels when fully loaded whilst the steel in the lower tensile chord should be fully stressed prior to prestressing. Furthermore, in order not to inhibit progress, encasement would be concurrent with construction of the upper floors.

The most economical sequence showed four steel trusses supporting the plant room plus four office floors above in their unencased condition. Encasement of the compression members was to be completed by this time and encasement and first stage prestressing of the bottom chord at this stage of plant room plus eight floors. This sequence was later modified to plant room plus three for encasement and plus five for prestressing to suit the contractor.

The truss members are two plates of maximum dimension 710 mm × 50 mm battened together to form 400 mm wide box sections. All joints were butt welded but provision made for fabrication in three sections for transportation with site connections made with 32 mm diameter high strength friction grip bolts.

The steel was specified as *BS 4360* grade 50B and was to be ultrasonically tested at the rolling mill for lamination.

Care was taken with joint and weld design to minimize lamellar tearing and distortion. Low hydrogen electrodes prebaked and plate preheating to 200°C was specified, together with an extensive programme of destructive and non-destructive testing.

After encasement full composite action of steel and concrete was assumed and transfer of load from concrete to steel was made via studs welded to the sides of the trusses and concentrated at node points. In addition, a helix made from 12 mm diameter mild steel was welded to each batten plate to assist bond between steel and concrete.

The end diagonal of the trusses carries the full load of the tier as a steel member and a finite element analysis was carried out to establish the stress distribution through the gusset plates at the junction with the top chord composite members.

When encased, the girder is 3 m wide with top and bottom flanges of 900 mm and 1000 mm depths respectively, the depth of the bottom flange being determined by spacing of the prestressing cables. We required 15 cables each with 79 wires but 18 were provided to give a safety factor against mishaps. This was later to prove a wise precaution as in one girder at the 35th floor level, two cables were rendered useless due to fracture of the metal duct during encasement.

Just prior to tender, world delivery time for steel looked as if it would lengthen dramatically and the client decided to supply the steel plate for the girders. We suddenly found ourselves with the added responsibility for providing cutting lists for this steel in the knowledge that if we made a mistake a likely delay of six months would result. Needless to say they were adequately checked before issue!

The girders span between mechanical bearings mounted in recesses in the core walls. We issued a performance specification for the bearings based on standard Proceq Reston PN and PNE types which were eventually used. They allow rotation at each end and translation at one end.

Loads are applied to the top of the girders via short steel box columns which also act to combine the four steel trusses into one prior to encasement. They are of 20 mm high tensile plate filled with concrete after installation. A small amount of grinding plus strip shimming was specified to ensure positive bearing on all four trusses.

We were anticipating movements of fairly large magnitude especially before encasement was completed and could not provide Freyssinet type hinges to accommodate them as we had done at Level 4. We considered using steel/neoprene type bearing pads but in the end simply freed the column from lateral restraint at the upper floor level giving an effective length of some 4.6 m which proved adequate.

During the design period we had made a survey of steel fabricators and had come to the conclusion that the only firms having the technology capable of fabricating the steel trusses to the standard we required were those engaged in shipbuilding and repair. We issued to the tenderers a list of approved subcontractors and we were fortunate that the firm we considered the best of them, Bethlehem Shipyards, a subsidiary of the giant American steel producer and fabricator, was chosen. In our opinion, the technical success of the trusses was in no small way attributable to their technical competence and know-how.

Their fabrication shop at Sembawang was very modern with all plate cutting and edge preparation done by automatic processes. All welding however was done by hand and all welds tested either radiographically or ultrasonically.

This testing was carried out by an independent body and if any test results were considered marginal they were sent to Australia for final arbitration. Very few welds were rejected for non-compliance which is in some measure due to the degree of testing specified and the engineering supervision provided by ourselves. In addition to a resident clerk of works, Bob O'Hea, who was the job engineer in Australia, was sent to Singapore for this purpose and later became resident engineer proper after Ron Bergin left in December 1974.

The trusses were fabricated in three parts for transportation and assembly but were completely test assembled in the shop before despatch.

Sembawang is on the opposite side of Singapore island to the city and trusses were transported by barge around the island to Singapore harbour about 0.5 km from the site. From here they were hauled on low loaders through the city and Old Chinatown and placed on a prepared bed in front of the building for assembly and final checking.

Assembly was made with 32 mm diameter high strength friction grip bolts using the part turn method. A full load test on a typical 42 bolt connection assembled this way had been carried out in the shop as part of the original testing programme. The trusses were stable under their own weight over the 35 m span but, as a precaution during lifting, stabilizing beams were added to the top chord.

The all up weight to be lifted was about 48 tonnes and trusses were winched to within 6 mm of their final position in about 45 minutes at both 20th and 35th floor levels. Small hydraulic jacks were used for final positioning and the complete operation went very smoothly.

Our construction sequence at tender was based on a particular sequence of encasement and floor construction and an assumption as to construction loads on the already cast floors. Both these parameters were variable and strain gauges were installed in order that we could monitor primary and secondary stress levels at specific critical locations in the girders both before and after encasement. We had hoped that this would enable us to approve or disapprove of any variation to the construction sequence without recourse to recalculation and would give us verification of our assumptions as to the behaviour of the composite members. Unfortunately, this proved to be of no value whatsoever, the readings being so variable as to be particularly misleading and the gauging was terminated.

For fairly obvious reasons the plant room floor and the floor immediately above the girder had to be self-supporting and were designed as *Bondek* floors spanning over steel beams. No propping is required for this type of construction and the *Bondek* sheets were fixed by shear studs welded through to the beams. Concrete was then placed and the two floors used as staging for the typical concrete tier built above. The steel beams were later fireproofed using a vermiculite concrete but this was not necessary for the slab proper.

In order to reduce weight on the plant room floors spanning 13.7 m and inhibit vibration, plant was mounted on thin secondary slabs themselves supported on *Silentbloc* rubber and steel mountings. These carried a maximum of 6.5 kN at about 1.2 m centres and allowed the secondary slab to be jacked up and floated on rubber pads, thus eliminating heavy plinths.

Floors at Levels 50 and 51

In order to standardize the three banks of typical floors, those at 50 and 51 were considered to be self-supporting between the service cores.

The original tender design allowed this to happen quite comfortably as the uppermost floor was an open plant room carrying water tanks and cooling towers with deep parapet walls. These walls formed the main supporting elements and, in addition to Level 51, carried Level 50 suspended from hangers.

The construction was of prestressed and reinforced concrete built off the 49th floor level.

When construction of the third tier was well under way, two changes occurred which radically affected the structure occasioning a complete redesign. The architect first decided that he could not tolerate any vertical structure between the floors so that the hangers had to come out and the client almost simultaneously decided the penthouse floor was too small and should be increased to typical floor proportions.

These decisions merely altered Level 51, but posed problems for us at Level 50. We adopted the standard terrace floor beam and slab layout but in lieu of columns provided a double tee beam 1.7 m × 2.3 m wide with 460 mm webs spanning between the service cores. These beams were too shallow to be efficient and were further complicated by a reduction in depth 3 m from the supports to 1.3 m to allow air-conditioning ducts to pass under.

The analysis of the floor was interesting. The deep edge beams are connected to the cores by short cantilevers and, with everything connected up, are stiff in relation to the main beams. Consequently, due to relative differential deflection, the three most central transverse beams try to span between the edge beams rather than over the main beams. We therefore reduced the stiffness of the edge beams by providing small discontinuity gaps in the upstand portion and disconnected them from their cantilever returns until all dead load had been applied, giving us a more satisfactory and determinate system for design.

The beams were cast onto neoprene/steel *Proceq* bearings and a complete section left out of the floor to accommodate as much of the prestressing movements as possible.

Because of their shallowness, the beams required a very high prestress and 10 (4000 kN) cables each with 79 wires were provided. The cables just managed to clear the notch in the beam soffit but we were not left with enough end surface for the anchorages so that four cables were terminated at the notch. The stress distribution at this point then became of concern as apart from anchorage stresses, we had to deal with those due to double curvature and change of section. A three-dimensional finite element analysis was carried out over this section to verify our hand calculations.

The beams were stressed in two stages of 80 per cent and 20 per cent with the application of floor screed and a time lag of two months between them.

Design of Level 51 was straightforward in beam and slab construction spanning between the parapet walls. These spanned between the cores and were prestressed using six 79-wire cables per beam. Releases and bearings were as for the 50th floor.

At one end of this floor there is a helipad above for use in emergencies such as fire, designed to Department of Civil Aviation requirements.

Cladding

The finish to the large exposed concrete areas of the building such as the service cores and transfer girders was not determined until $1\frac{1}{4}$ years into the construction period. We had simply allowed for a load equivalent to 50 mm of concrete as it was originally thought that a rendered type finish was most likely.

After considering various precast concrete, stonework and ceramic alternatives, the clients decided to clad the building with white Sardinian granite.

Fig. 15
Granite cladding to the core
(Photo: Kouo Shang-Wei)

Tenders, which were to include the method of fixing, were invited and we were employed under a separate commission to act as proof engineers for the system chosen and to supervise installation.

The granite was applied as 1.22 m × 300 mm × 25 mm thick panels supported by stainless steel anchors grouted into holes drilled into the concrete surfaces. A nominal clearance of 25 mm was to be allowed between the concrete surface and the panels, and a 6 mm gap between panels to allow for movement. This was to be sealed not so much to make the system waterproof, but to inhibit growth of plants from seeds dropped by birds, a reasonably common phenomenon in Singapore.

The basic anchor, manufactured in Germany by Lutz, consists of a strip of metal 140 mm × 17 mm × 3 mm thick, twisted at one end to create a flat on which to bear the panels and at the other to form a fish tail. Through the flat portion passes a 4 mm dowel which stabilizes panels horizontally both above and below the anchor, the lower dowel passing into a plastic insert in the panel, thus allowing easier movement.

A programme of testing of panels, anchors and grout was carried out and as could be predicted, the system was based more on practical than theoretical consideration, the minimum factor of safety being in the order of 10.

The grout chosen was *Embecon 603* for its non-shrink properties and pot life after mixing. We were not concerned with the effect of its ferrous constituents on the anchors and the possibility of staining was eliminated by coating the exposed surface with epoxy.

The concrete walls had not been designed to accept any form of cladding system and a whole range of alternative fixings had to be designed to cater for those occasions when reinforcement would be struck whilst drilling holes for the anchors. This was a particular problem at the bottom of the core walls and at prestressing anchorage locations, but did not create anything like the trouble we had anticipated overall. We produced a manual of procedure which anticipated most difficulties encountered.

Although the contract documents had assumed the cladding to have a geometry of its own independent of the concrete surfaces, this proved impractical in terms of setting out and construction speed and was abandoned – the granite simply following the concrete profile and maintaining the 25 mm clearance.

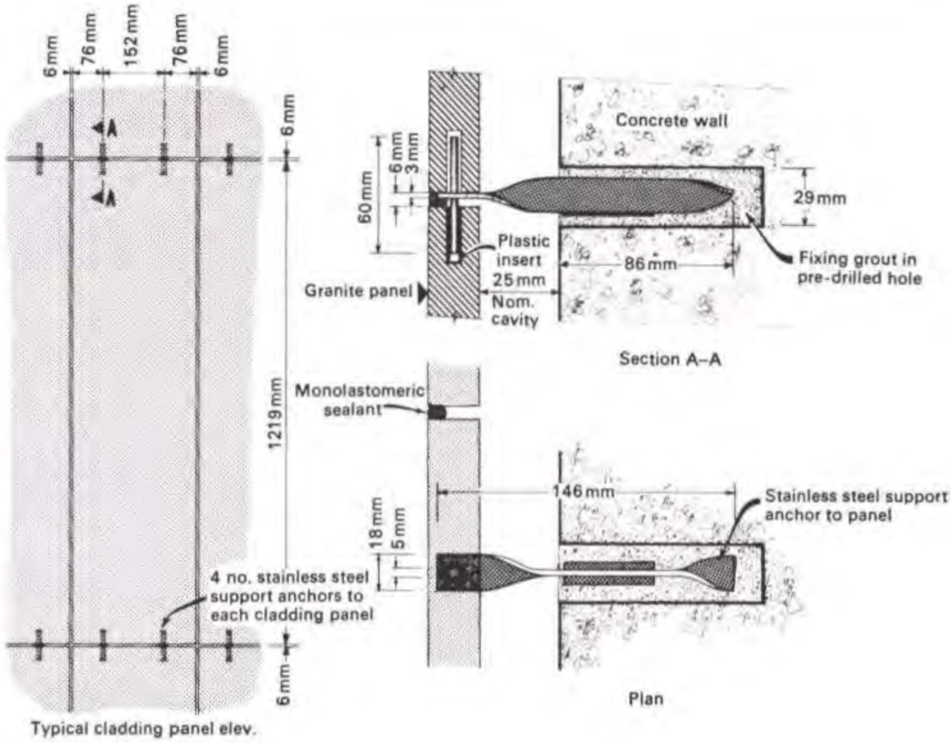


Fig. 16
Fixings to granite cladding

Fortunately, the core walls were within reasonable tolerance and without any excessive 'lumps' so that the end result was quite satisfactory.

The construction sequence was quite rapid with two rows of panels fixed at each core per day from hanging stages. The holes would be drilled and the panels loaded onto the stage overnight and the panels set by day; each anchor was tested in situ for a minimum pull out of 25 kN after four hours.

The 6 mm gap between the panels was sealed using *Tremco Mono*, this being the only sealant practicable for this application. At plaza level, where fingers could be poked into joints, a sister product, *Dymeric*, was used which becomes tougher a little quicker than the *Mono*.

As of August 1976, the building is complete apart from a few granite panels yet to be installed and cleaning up of the upper tier of floors and plaza area.

The banking hall is occupied and the first tenants moved in. Construction has taken about six months longer than originally planned but is still within the time estimated for conventional floor-by-floor construction, which would possibly have over-run by a similar amount.

Credits

Client:
OCBC Centre (Private) Ltd.
Architects:
I. M. Pei & Partners (New York) in association with BEP Akitek (Singapore)
Main contractor:
Joint venture – Morrisen-Knudsen (USA) and Low Keng Huat (Singapore)

Traffic engineering in Australia

Jon Burgmann

The firm first became involved in traffic and transportation studies in Australia about six years ago. Since then we have developed our skills and reputation in three main fields.

1 Master planning and development control plans – most commonly undertaken for institutions or planning authorities such as local government
2 Feasibility studies and project planning – usually carried out for private development clients, but also for government construction authorities
3 Expert advice and evidence in planning disputes – where we have advised government and private clients with approximately equal frequency.

By their very nature, the first two categories of our work are almost always undertaken as part of a multi-discipline study involving other professions such as planners, sociologists and economists. Often civil and structural engineering services are also provided by the firm. We believe we now enjoy a modest reputation for approaching these projects in the best Arup tradition of combined team work.

NORTH SYDNEY AND MOSMAN DEVELOPMENT CONTROL PLANS

Historically the North Sydney Control Plan was the first major transportation commission awarded to Arups in Australia. The Mosman study followed later, but it is convenient to consider the two adjoining municipalities together.

The central business district of Sydney is on the southern shore of Sydney Harbour. Across the Harbour Bridge on the northern shore lies the municipality of North Sydney through which all movement bound for the Harbour Bridge must pass. In one sense, North Sydney functions as a giant transportation funnel for all north-south movement via the bridge, which carries 10,000 vehicles in the peak direction in peak hour, plus rail traffic.

The problems evident in North Sydney, which led to appointment of a planning consortium in 1971, were basically twofold.

1 Congestion on approaches to the Harbour Bridge (see Fig. 1) was extensive and impeding access to North Sydney itself. In addition, in an attempt to avoid the congestion on the main traffic arteries, commuters had developed an elaborate series of 'back routes' resulting in excessive through traffic in residential areas.
2 North Sydney at that stage was already recognized as a developing commercial centre. The 1971 workforce of 16,000 was projected to grow to 50,000 in the early 1980's (already 30,000 by 1975). This rapid growth, together with more intensive redevelopment of residential areas for high-rise home units, was producing pressure on available on-street parking and community facilities in general.

The problems of Mosman were not quite the same in that commercial development was more modest. However, Mosman was experiencing a similar expansion of residential population due to redevelopment for high-rise living in the attractive harbourside suburbs. In addition, Mosman was bisected by the major transportation corridor from Sydney's northern beaches area to the city. Commuting traffic was utilizing a multitude of residential streets as alternative routes. A planning consortium was appointed to develop a planning and transportation policy for the municipality late in 1972.

Fig. 1
Morning peak hour in North Sydney on approach to Sydney Harbour Bridge

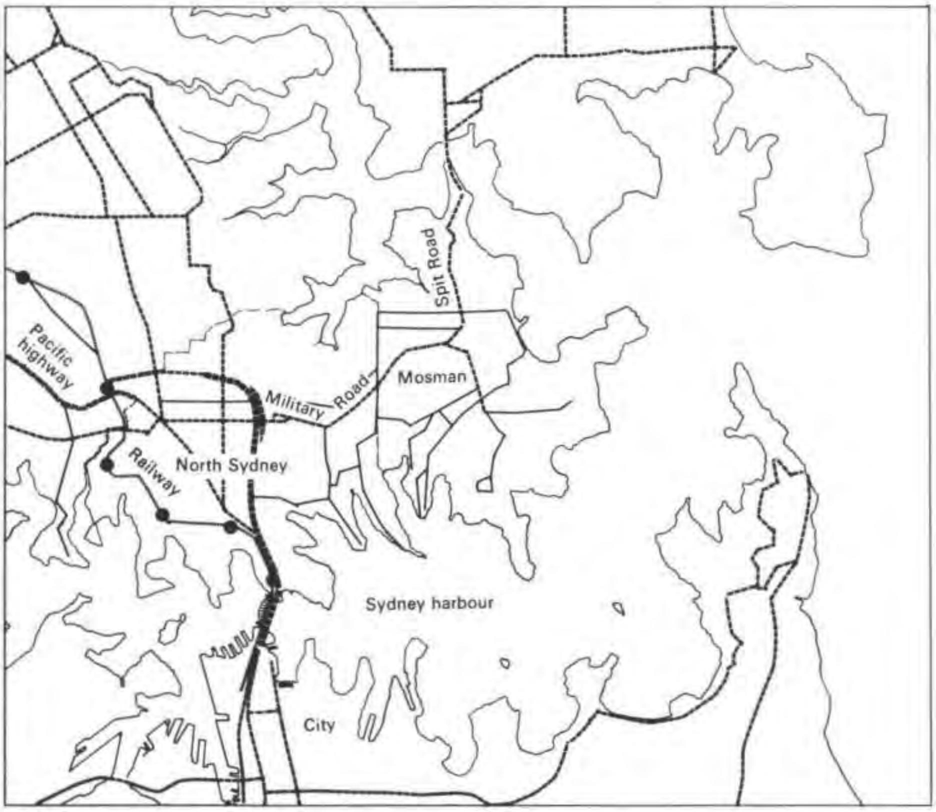


Fig. 2
North Sydney and Mosman – primary road network

The relation of the two municipalities to the primary transportation network is shown in Fig. 2.

Surveys undertaken concentrated on the weekday peak period and were aimed at determining comprehensive travel characteristics of the municipalities. They included cordon surveys (with police assistance), home interviews, public transport patron surveys (including the 12 ferry wharves in the two municipalities), detailed road counts, parking studies, and a review of accident statistics. An important aim of the data collection was to allow differentiation between through traffic and traffic with a local destination or origin. The proportion of public and private transport for the different trip types (i.e. modal split) was considered important as it was felt that in this area some alleviation could be made to the pressure on available road space. Fig. 3 shows diagrammatically the modal split and distribution for the journey to work of the North Sydney workforce, June 1971.

Having completed our data collection and analysis phase, the whole planning team addressed itself to the task of formulating policies. It was obvious from the very beginning that certain constraints and assumptions would have to be accepted. For instance, the existence of the Harbour Bridge and its regional role could not be denied, and the likelihood of an additional harbour crossing in the near future was remote. Financial constraints meant that little or no land acquisition could be contemplated. The ability of either council to change travel characteristics (such as modal split) for the through traffic in their areas was restricted to any influence that could be brought to bear on the various state planning or transportation authorities. However, by control of development codes, parking restrictions and local street systems, each council could alter the travel patterns of its own residents, and those whose work place was within the municipal area.

At this stage we had our first contact with 'public participation'. After a presentation of the transportation data to each council, at

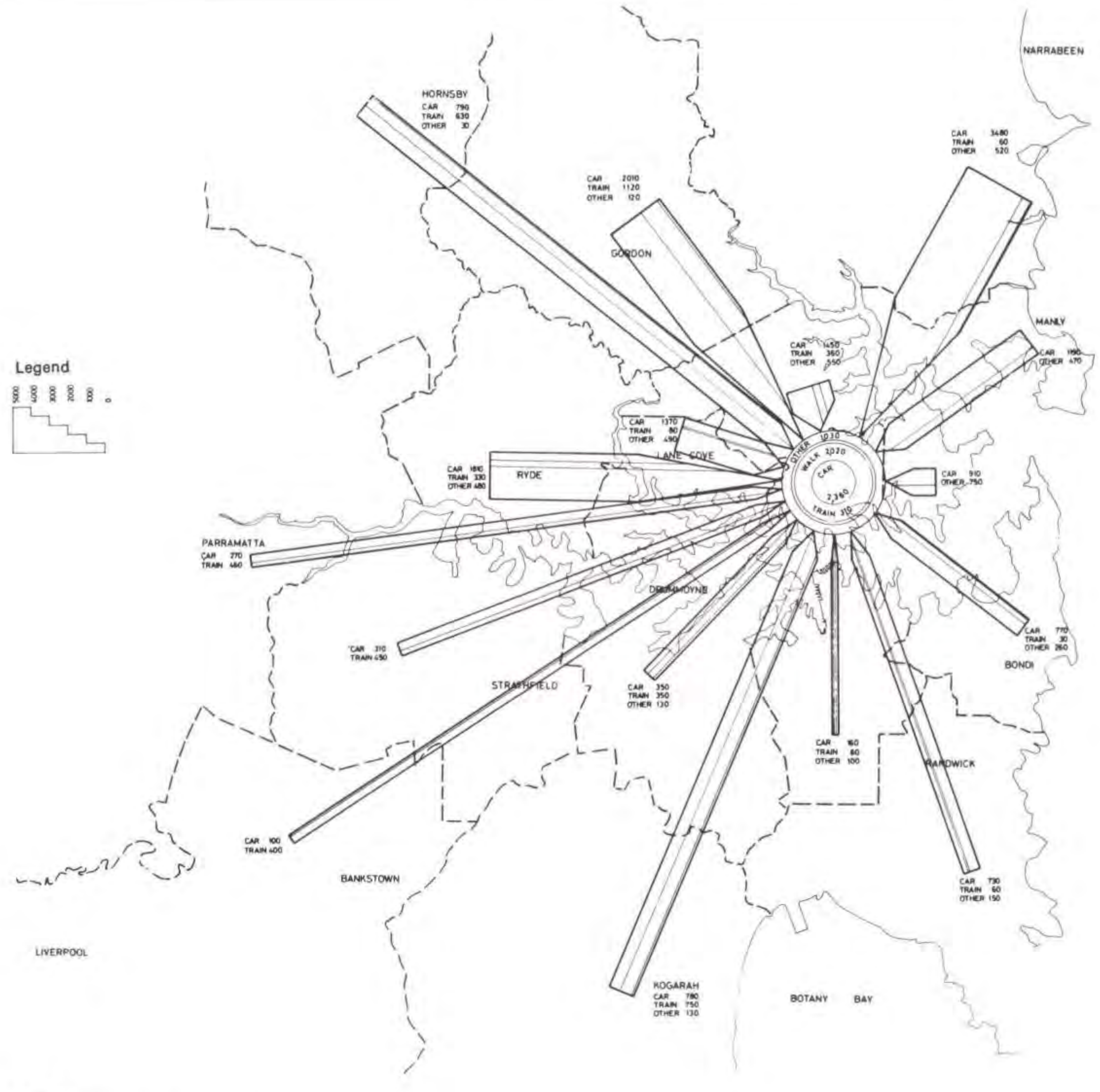


Fig. 3
Morning movements into North Sydney

which problem areas were also highlighted, the whole question was opened to the public – rate payers, progress associations, chambers of commerce, etc. In North Sydney particularly, all consultants were involved in extensive meetings with local groups, often lasting late into the night.

We found that, whereas few lay members of the community understood, or cared about floor space indices or daylighting factors, nearly everyone drove a car and considered themselves knowledgeable on the subject of traffic. Hence, we found a large proportion of the suggestions and criticisms directed at our part of the team. Fortunately, we had a very alert study director who chaired all such meetings and he managed to field most of the curly ones before they slipped through to us.

There was strong resentment expressed by local residents to commuter parking in residential streets and through traffic in residential areas. This concern was to spark major public controversy later at the conclusion of the Mosman study.

Participation by the public and genuine public interest continued through all stages of the studies to adoption of final planning proposals. From the technical point of view it is doubtful if it really contributed significantly to our planning efforts, except perhaps to draw attention to some sociological aspects.

Public participation did, however, play an important role from the client's point of view. Doubtless it enabled the residents (the electorate) to let off steam and to a certain extent be deceived into believing their views had been accepted. However, it is probably not too far off the mark to say that the local aldermen did recognize the rate payers' feelings, and in the final analysis it was these aldermen who accepted or modified the planning proposals.

From projected commercial, residential and retail development, using trip generation data from the surveys, total movements were obtained in and out of the study area. A determination was made of the cordon capacity of the primary road network. We then sought to limit total vehicle movements to a level compatible with this capacity, by adjustment of the modal split for various corridors. As far as it was within the power of councils to act, this was to be approached by control of parking (on and off street) and improvements to public transport. In this latter regard, councils could only use persuasion.

A very strong case was developed for the use of bus-only lanes on Military Road and Spit Road, which form the main corridor from the northern beaches through Mosman and into North Sydney (see Fig. 2). It was shown that this action would improve average journey times and increase total capacity of the system. In addition, by making bus scheduling more reliable and faster than private cars, more commuters would be attracted to public transport.

After much debate the responsible state traffic authorities introduced a pilot scheme on part of the route. It was modified slightly to allow taxis and private cars with a certain minimum occupancy to use the bus-lanes also. Subsequently the pilot scheme was judged a success and has now been extended.

When we were satisfied that private vehicle supply could be controlled and the excess demand met by other modes, it was still necessary to examine movements within the area. There were certain notorious locations where congestion or accidents were a continual problem. These were handled by various traffic management techniques.

More taxing was the problem generated by all those drivers who knew (or thought they knew) of a way around these trouble spots: that is, the problem of through traffic on residential streets. This was not a case of inadequate capacity, but an environmental and moral issue; also hotly political as local government elections approach at the end of the North Sydney Study.

It became an adopted policy in each study to eliminate or reduce through traffic from the residential precincts. To achieve this aim, a detailed study was necessary at each location to determine the best combination of street closures and traffic control devices (turn bans, one-way streets, etc.). The tricky part was to discourage commuters without making access untenable for local residents. Who is a commuter anyhow? To some people, anyone who didn't actually live in their street was an outsider, a commuter.

Before any significant action could be taken with street closures, it was essential that the primary distributors had the capacity to carry the flow diverted from residential areas. We proposed a staging sequence whereby sections of the primary network could be upgraded in capacity (re-phasing signals, parking bans, realignment, etc.). Extensive lobbying and negotiations by the council and ourselves was required because such main roads are under the control of the DMR (state road authority). Further, traffic signals are controlled by another state government department, whilst police and public transport concurrence was required in any proposal affecting their interest. When primary network capacity was provided, then it was proposed that street closures could be introduced to exclude extraneous traffic from residential precincts.

It was at this point that politics took over. The aldermen from both councils came under extreme pressure from local residents to act on street closures immediately, even before improvements to the primary network could be started. The councils did proceed with street closures: North Sydney first, and Mosman six to nine months later. For some days on each occasion there was turmoil, and genuine traffic chaos.

Motorists who had used a route for years felt incensed and either ignored signs or drove over the small parklets used to form a 'closure'. North Sydney and Mosman Council each responded with more substantial barriers and chains set in concrete posts (see Fig. 4). Still the onslaught of determined motorists found chinks in the defences and poured through. It was like a city under siege. The police said it was illegal but appeared reluctant to act. One Minister decried the action, another applauded the rebuttal to the great god car.

Each time the controversy raged for several weeks, and our humble traffic proposals became the subject of banner headlines, editorials, newscasts, court actions and questions in the House. Even the cartoonists entered the fray such as the Benier sketch in Fig. 5, showing an intrepid commuter beating the closures in his own private tank.

A selection of the headlines reads:

BLACKFRIDAY TRAFFIC – ROADS CLOSE

DRIVERS ANGER AS ROADBLOCKS GO UP

Fig. 4
Road closure in Cremorne, North Sydney
(Photo: John Fairfax & Sons Ltd.)

Fig. 5
Benier on road closures
The Sun newspaper, Sydney, 20 October 1972

"Darling — you're home early . . ."

COUNCIL BARRICADE
TORN DOWN BY YOUTHS

NORTHSIDE ROAD CHAOS –
GOVERNMENT IN NEW MOVES

CABINET ACTS ON ROADS

VICTORY ON STREETS
MAY CHANGE LAWS

HIGHWAY ROBBERY –
37 SHUT UP STREETS

MOSMAN COUNCIL STOPS THE TRAFFIC

North Sydney predominantly gained victory through the courts for the trial closures they wished to maintain. Mosman, however, was more ambitious and the closures both more extensive and radical.

After protracted argument, Mosman Council finally agreed to remove the barricades following reassurances from the Minister that the DMR would bring forward its programme for upgrading Spit Road and Military Road. Perhaps it was all a clever ploy by Mosman Council to provoke action from the state authorities concerned.

Most of the residential precincts in both municipalities have now been 'protected'. Major improvements have been effected to the primary network following our guidelines, and we have a small continuing involvement in implementation of the proposals for North Sydney. Also, as with any planning for a non-static situation, there is need for monitoring and occasional adjustment but generally the system is functioning satisfactorily.

The North Sydney Study was one of the very first major transportation plans undertaken by consultants for a local government body in Sydney.

Undoubtedly both the local planning profession and we as a firm gained tremendous experience from these two studies. This is particularly true with regard to public participation, interaction with state government transport authorities and the pressures within a body of citizens known as aldermen or councillors.

We have since been able to apply our experience to other studies in Sydney, Melbourne and elsewhere. However, none have again provided us with quite the drama of headline news, editorials and the cartoonist's barb.

Credits

Clients
Municipality of North Sydney and
Municipality of Mosman

Planner
North Sydney:
North Sydney Planning Consultants (Ancher Mortlock Murray & Woolley, and George Wellings Smith in Association)

Mosman:
Wellings Smith and Byrnes

Fig. 6
Moonee Valley Racecourse on raceday

MOONEE VALLEY TROTTING INVESTIGATION

One of the most unusual traffic engineering projects we have undertaken was in connection with a recent planning dispute in Melbourne. The investigation involved a detailed examination of traffic associated with trotting and horse racing. At the time it seemed to us that the whole commission was undertaken at a brisk gallop.

Trotting (the harness sport) is a popular spectator sport in Australia. Meetings are generally held at night and the drum of hooves and the colourful silks of the drivers create a festive atmosphere under the floodlights. A wide range of refreshments add further to the gaiety of the evening outing and betting facilities are heavily patronized.

Night trotting in Melbourne is currently held at the Royal Showgrounds. The actual track dimensions at the Showgrounds are substandard and the Trotting Control Board is seeking improved facilities.

The Moonee Valley Racing Club has proposed that a new trotting track be constructed within the existing horse racing track at Moonee Valley. This new track would be to the desired international standards. The proposal has further attraction because racing is a daytime sport and trotting a night activity. This would enable trotting and horse racing to share the use of many of the important facilities: catering, betting, stabling, parking, etc. The Moonee Valley Course has provision for over 9,000 off-street parking spaces, which can be seen in the aerial photo of Fig. 6, taken at a typical race meeting.

The firm was commissioned by the Moonee Valley Racing Club in 1975 to investigate the traffic implications of the proposed change of venue for night trotting. Extensive field surveys were undertaken to derive the trip generation characteristics and parking demand of night trotting, and to establish base flows in the Moonee Valley area.

It is fair to say that there was considerable interest and a degree of pleasure associated with some of the field work. We caught the air of excitement as we mingled with the race crowds, heard the starter's call over the loudspeakers and the roar of spectators at the finish.

A detailed survey of a trotting meeting at the Showgrounds was undertaken on a Saturday evening when approximately 15,400 persons attended.

A complete cordon was maintained around the Showgrounds and arrival and departure distribution recorded. Modal split was observed (both public and private), including special attention to the branch rail line serving the Showgrounds. On and off street parking was also recorded, together with spot checks on vehicle occupancy. A similar survey was undertaken at Moonee Valley for a Saturday race meeting, at which attendance was approximately 25,700.

A Saturday evening was examined at Moonee Valley to obtain base flows and base parking in the surrounding street network. From analysis of the trotting survey and adjustment for public transport facilities, the anticipated trotting traffic was derived for the Moonee Valley location. This was superimposed on the base counts to obtain predicted flows and parking.

These figures were analyzed in terms of traffic flow, intersection capacity and extent of parking. The projected figures were also compared with the day-time racing figures so that the data and implications could be more readily interpreted by lay persons. It was generally shown that conditions would be less congested than those associated with race meetings and were acceptable in traffic planning terms.

A detailed report was prepared and evidence given before a lengthy public hearing of the Town Planning Appeals Tribunal. Other contentious issues which the Tribunal had to consider related to amenity, lighting and noise. The findings of the traffic study were challenged in the usual way but generally the validity was not in dispute. We are pleased to note that planning permission has now been granted.

Credits
Client
Moonee Valley Racing Club

The CBA banking chamber dome

Erik Guldager-Nielsen

Melbourne has some very fine buildings constructed in the late 19th century. John Betjeman, during a visit to Australia for a television series, described them as some of the finest buildings of the Victorian era anywhere in the world. Many, like State Parliament House, have ceilings inlaid with gold leaf and record the hopes and aspirations of the people as Melbourne changed from being the single-storeyed capital of a gold rush state to a stable modern city supported by agricultural and trading wealth.

The banking chamber of the Commercial Bank of Australia is a great Victorian interior.

The CBA has had its headquarters at 335 Collins Street, Melbourne since 1866. Spurred on by the land boom in the 1880's, the Bank commissioned the architects Lloyd Taylor and Alfred Dunn to design a new head office after a competition; the selected design was a compromise between individual entries by the two architects. In spite of great economic difficulties caused by the collapse of the land market in 1890 the present banking chamber was erected in the years 1891 to 1893.

The building then consisted of a neo-gothic Collins Street façade, the octagonal domed banking chamber and a rear building with additional office space. In 1939 the Collins Street façade was demolished and a new frontage in the heavy style of the 1930's was erected, adding two floors to the building, thus creating more office space. At the rear and also on the Collins Street frontage, neighbouring properties were acquired and extensive alterations made to connect the different buildings. This resulted in a maze of outmoded and inefficient space, some since subject to orders of fire protection authorities, with the original banking chamber standing in the middle.

To accommodate future growth, the bank decided in the 1960's to redevelop and by the time Ove Arup and Partners became involved as consultant structural engineers for the proposed new multi-storey office block, it had been decided to demolish the banking chamber to make room for the new development.

The banking chamber is an octagonal space, 18.3 m across, and the ribbed dome rises to 18.3 m above the ground level; above this a lantern rises another 9.15 m. At the time of construction it was described as 'the largest dome in the southern hemisphere', and, further, that it was unique in so far as it was 'constructed solely from plaster'.

When the news of the intended demolition of the dome was announced, following an application for planning permission, there was considerable public protest. This was made through the media, by public petitions and by comment in State Parliament. It was at this stage that the National Trust announced that it was essential that the dome interior should be preserved, and gave it its highest classification. Strangely, although many other buildings in Melbourne had been so classified previously, the CBA dome in the heart of the city had been overlooked.

The Victorian Premier decided to set up an Enquiry to investigate the architectural and historical significance of the chamber and to recommend whether or not it should be retained.

Ove Arup and Partners were commissioned by the Bank to carry out a full investigation of the dome to establish details of materials and construction, the present structural state of the dome and any restraint that would be imposed on the future development of the remainder of the site.

Fig. 1
Interior of the CBA banking chamber
(Photo: Ian Mckenzie)

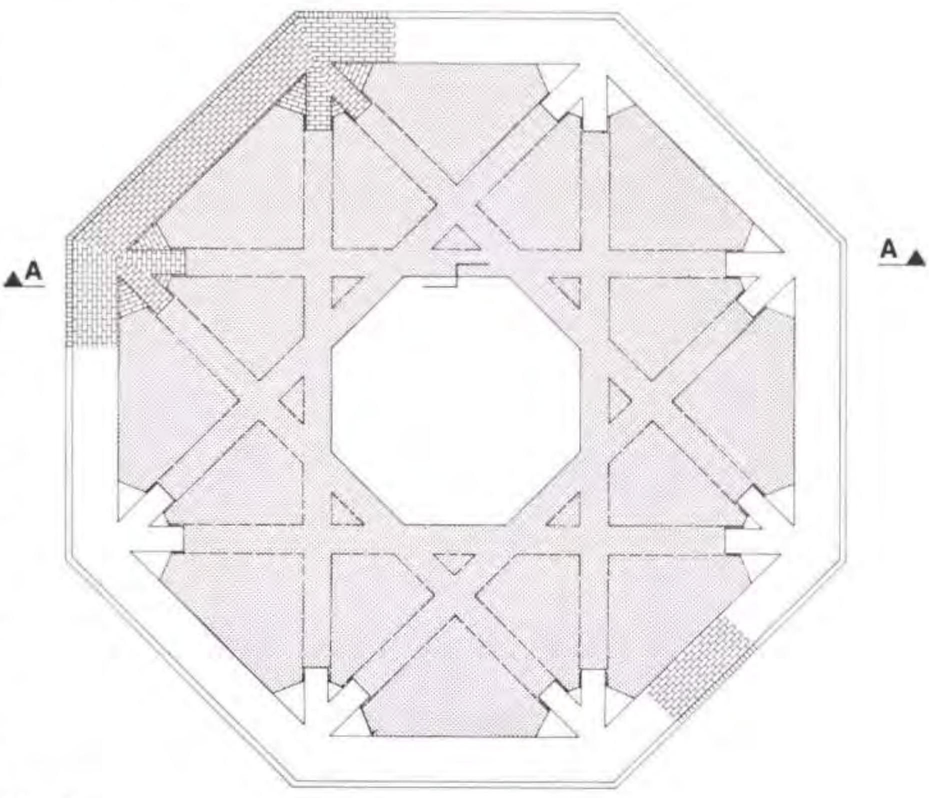


Fig. 2
Structural arrangement of dome and ribs with the opening for the lantern in the centre

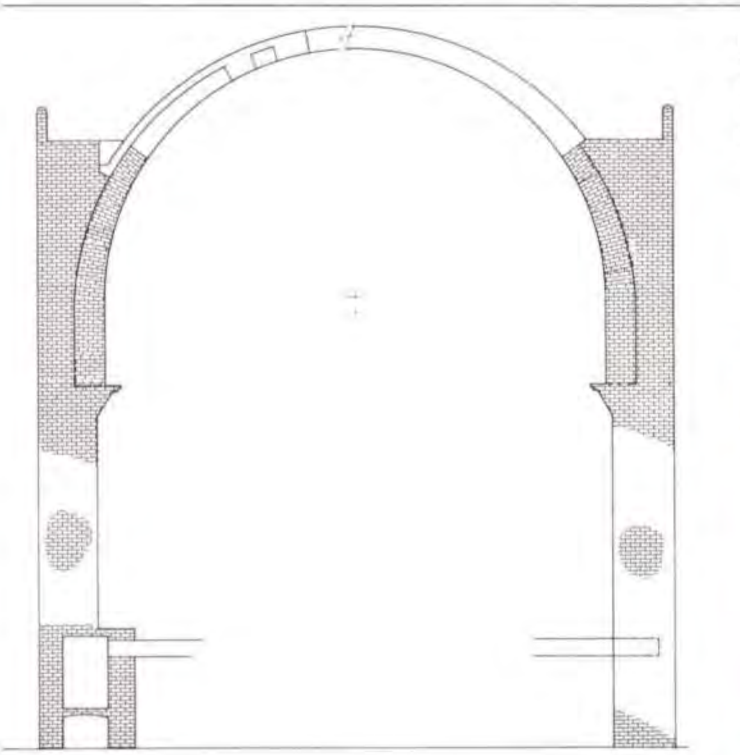


Fig. 3
Cross-section through the dome

Fig. 4
The lantern from below (Photo: Erik Guldager-Nielsen)

Fig. 5
Detail of plaster work (Photo: Erik Guldager-Nielsen)

The time available was short, and careful planning was required to minimize the interference to normal banking operations. During the course of one weekend samples were drilled from the structure and holes were opened to see whether any steel ties had been incorporated in the construction. Test pits were dug in the basement and samples taken from the footings and the supporting soil.

The dome consists of brick piers and columns up to 1 m below parapet level; above this the entire dome is built from concrete, a weak concrete, more like a mortar with very little coarse aggregate and no reinforcement at all. Laboratory tests on the samples showed that the concrete was very porous. The density is only 1650kg/m^3 and the compression strength about 10 N/mm^2. It appears that the concrete in the bottom of the ribs is stronger and contains more coarse aggregate, obviously a result of segregation (in this case beneficial) and better compaction.

Chemical analysis showed that the concrete was sound and the assortment of aggregates used – basalt, scoria and broken bricks – would not cause chemical deterioration of the concrete.

During an extensive crack survey numerous structural cracks were found. A computer analysis was carried out assuming the intersecting ribs formed a rigid jointed framework. This showed the observed cracks were in positions of theoretical high torsion stress but there was no evidence of excessive bending which might endanger the structure. The cracks could be measured only on the top surface of the dome and were found to be from 0.2 mm to 1 mm wide. Larger cracks had opened up between the dome concrete and the surrounding brickwork; one of these was measured at 17 mm, but the position made it structurally insignificant.

The lantern is wrought iron, quite badly corroded in places; the supporting walls and columns are very solid brick work, and the foundations are adequate by modern standards, although it is evident that some differential settlement has taken place as a result of adding two floors to the front of the building. Measurement of cracks in footings and external walls indicates that this settlement could be about 4 mm. Further differential settlements were simulated on the mathematical model, which showed that the dome is very sensitive to differential movements and consequently great care needed to be taken if deep excavations were to be carried out immediately adjacent to it.

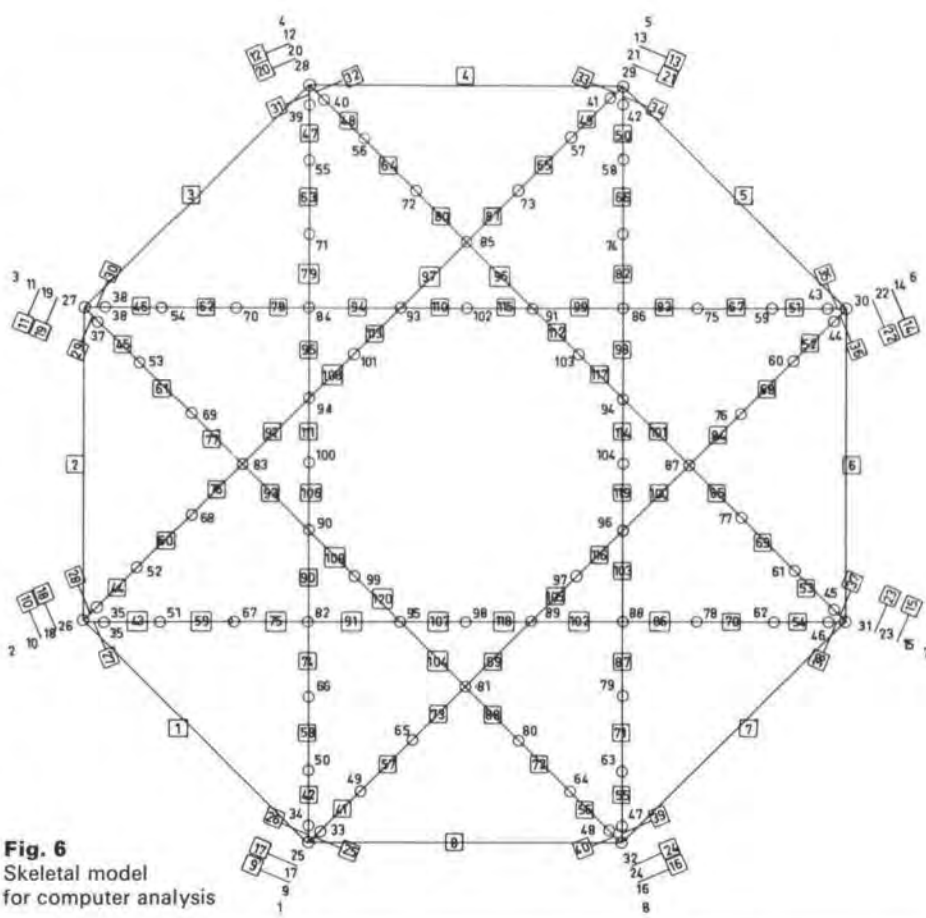


Fig. 6
Skeletal model for computer analysis

We presented all this evidence to the Enquiry, being able, on the Bank's behalf, only to stress the fragility of this kind of structure and the limitations this would impose on any construction work adjacent to the dome.

The Enquiry duly found that the chamber interior must be preserved. The Bank is now planning for redevelopment of its site with retention of the dome in a manner which will integrate it into the new building with elegance.

Credits

Client

The Commercial Bank of Australia Ltd.

Australian embassies

Peter Thompson
Peter Haworth

INTRODUCTION

In 1973 the Australian Government embarked on expansion of Australian representation in foreign countries.

Historically speaking, Australia had in many countries either been represented by, or housed in, the British Embassy and in most instances relied on the British for specialist services such as communications.

With the lessening of ties with Great Britain it was felt that Australia's image abroad should be improved and appear more independent. A new government department, the Overseas Property Bureau, was set up to be responsible for all government property abroad, and to implement the policy that where overseas offices were housed in rented accommodation, they were to be relocated in Australian Government-owned premises. This meant, in effect, the construction of chancery buildings in many countries.

The first commissions for these new buildings were given to private architects, and we were fortunate in being invited to participate in four of them – our Melbourne office with Singapore and Kuala Lumpur, Sydney office with Bangkok, and Perth office with Saigon.

Construction is now completed in Singapore and well under way in Bangkok and Kuala Lumpur. The fate of the Saigon Chancery needs no explanation, it eventually becoming a race in time between finishing the working drawings before the government told us to stop, and the Viet Cong taking over!

The need to cut government spending in Australia has meant a slow-down in this area, and furthermore, a special government design department has been set up to service the Overseas Property Bureau's needs. Although conceived as a multi-discipline group, they do call upon private consultants to assist them in various aspects of their work. For example, we have been commissioned to prepare the structural design for a prefabricated aluminium-framed house for the Australian High Commissioner on the island of Nauru.

AUSTRALIAN EMBASSY, BANGKOK

Bangkok is the largest of the projects we have been associated with to date, including not only a chancery but a residence for the Ambassador and staff accommodation. Ken Woolley of Ancher Mortlock Murray and Woolley Pty. Ltd., architects, was commissioned to undertake this project.

The site of 1.2 ha is located in South Sathorn Thai Road, Bangkok. The overall plan of the compound is based on a chancery block which is a hollow square in plan, placed in the front of the site with the official residence at the rear. The buildings are placed so as to preserve the major trees with the largest located within the courtyard of the chancery block.

As part of the landscaping the site is to a large extent flooded and the buildings stand in water, the site becoming a series of islands linked together by causeways at general road level.

The chancery accommodation is largely on three elevated levels with the main entry/foyer floating at courtyard level. Car parking and staff amenities are built partly into the ground on one side only.

Two of the upper levels are offices, the third being for accommodation.

Bangkok lies on the bank of the Chao Phraya River and the subsoils are Quaternary deposits consisting of alternate layers of uncemented sands and fine gravel, sandy clay and fine sand. These layers are overlain with thick homogenous clay deposits.

The site investigation showed 26 m thick layers of clay ranging from very soft to very stiff with respective unconfined compression strengths of 0.01–0.4 N/mm². At this level there is a thick deposit of medium to dense sand, having an average blow-count of 30 per foot.

All structures, including those at ground level, are supported on precast concrete driven piles ranging from 400× 400 mm, 27 m long driven into the sand, to 200× 200 mm, 19 m long acting in friction. Allowable working loads vary from 80 tonnes to 25 tonnes.

Preliminary pile tests were carried out as a supplement to our site investigation and indicated that we would have load factors of at least 3 on our assumed working loads.

All piles terminating in individual pile caps are raked to provide adequate lateral stability for their columns above, which in many cases rise 13.5 m to the floor above.

The car park and amenities block is partly submerged and is of conventional reinforced concrete beam and slab construction. The basement portion is tanked. This part of the building and the entrance foyer are connected by short bridges to the island courtyard which acts as a visitors' car park and is an approximately 180 mm thick reinforced concrete flat slab on piles at 3 m centres.

The three main floors and roof to the chancery block are generally supported on a 12× 12 m grid of 0.8 m diameter columns. They consist of diagonal interlaced beams of reinforced concrete 1 m deep× 340 mm wide at 6 m centres, a 120 mm slab filling the space in between the beams.

The floor slabs are staggered on plan with maximum cantilevers of 6 m occurring at Levels 3 and 5. The external periphery of Level 3 is suspended by a prestressed concrete hanger from the Level 4 diagrid. The structure is completely exposed at Level 3 and parts at Levels 4 and 5. The slabs were designed as grid frameworks.

The roof structure consists of structural steel trusses supporting aluminium roof sheeting. At 5th floor level, which is for domestic accommodation, there is a dual structural floor

Fig. 1
The Bangkok Embassy: entry
(Photo: David Moore)

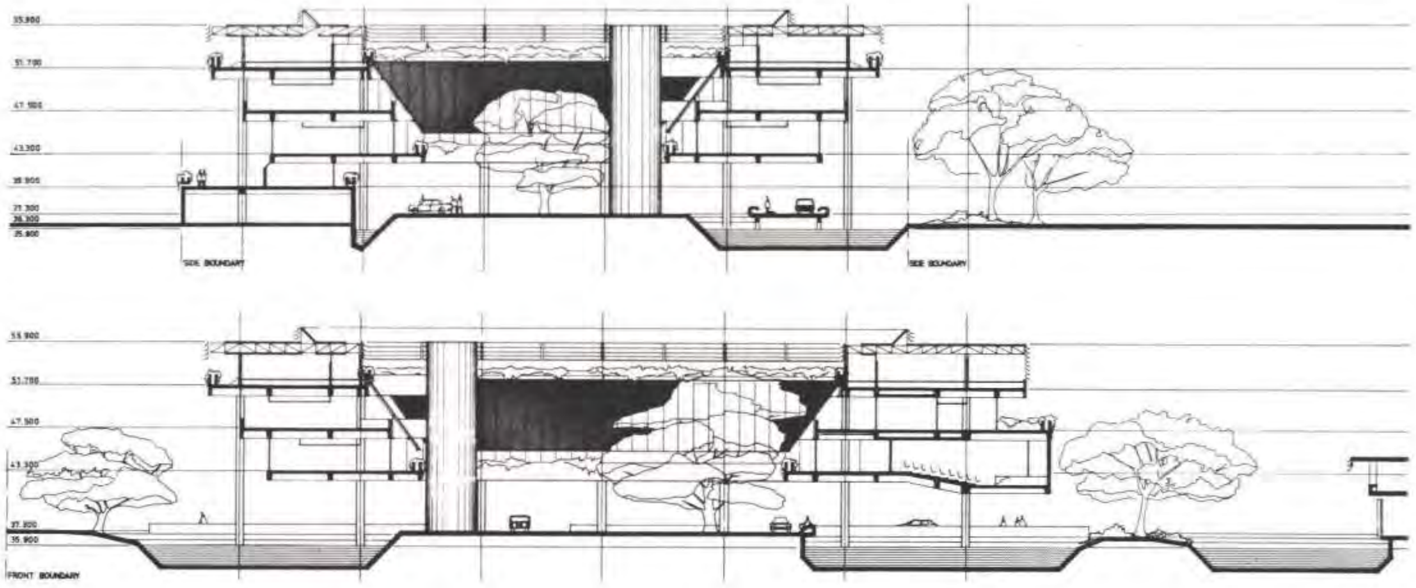


Fig. 2
Bangkok Embassy: sections

Fig. 3
Bangkok Embassy: view from the front (Photo: Max Dupain)

system – the upper being lightweight steel/concrete supported from the lower diagrid. This is to allow the falls necessary for plumbing and to give potential for eventual conversion to office space should the need arise.

The Ambassador's residence is a two-level structure with attached servants' quarters and a swimming pool. This is of conventional construction on piles.

Exposed concrete surfaces to both buildings are clad in either local stone or with glazed tiles.

The water landscaping involves considerable excavation to a maximum depth of 5 m. Where excavations are vertical, sheet piling of concrete posts and timber planking infill is used.

Concrete quality in Thailand is good compared to many other countries in Asia, and the use of ready-mixed concrete has increased especially in Bangkok proper, although the almost permanent traffic congestion does create problems. Concrete with cylinder strengths of 40 N/mm² can be produced, but we limited our requirements to a maximum of 30 N/mm² to ensure consistency. Reinforcement, however, is of inferior quality, being made locally from imported scrap iron, and it does not conform to the requirements of the Australian Codes of Practice and is available only in sizes up to 28 mm.

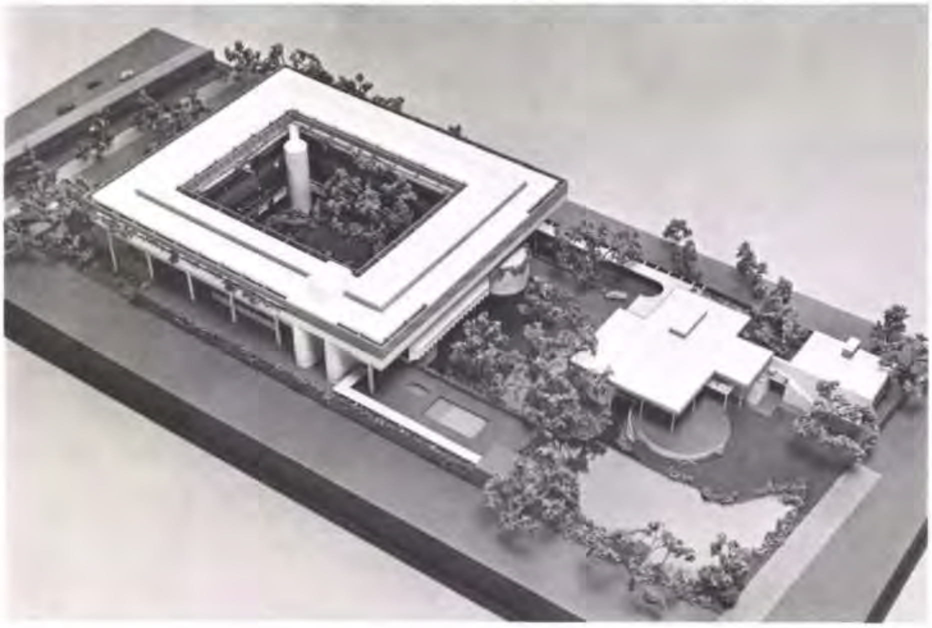


Fig. 4
Bangkok Embassy
(Photo: David Moore)

Because of the limitations of their locally-produced bars, ultimate strength design is at present not allowed in Thailand. This would have created problems with the design of our long-span upper floors and it was decided that local steel would be used for all construction up to ground level, and steel imported from Australia for the remainder.

Local byelaws require that a wind pressure of 1500 N/m² be used in the design regardless of positions or size of building. This figure is quite inconsistent with recorded wind velocity and a figure of 600 N/m² was used.

Construction is now well under way and although building techniques are fairly rudimentary and very labour intensive, Thais take a great pride in their work at all levels and the standard of workmanship is very high. This must be one of the few Arup sites in the world where the contractor has three engineers with MSc degrees as site engineers.

Credits

Client:
Australian Commonwealth Government

Architect:
Ancher, Mortlock, Murray & Woolley Pty. Ltd.

Main contractor:
Union Development Co.

Fig. 5
Singapore High Commission
(Photo: Ian Mckenzie Photography Ltd.)

AUSTRALIAN HIGH COMMISSION, SINGAPORE

The site for the Singapore High Commission was acquired in 1973 but with the special condition imposed by the Singapore Government that unless the new building was completed during 1976, the land would revert to local government ownership. Godfrey and Spowers Pty. Ltd., architects of Melbourne and Singapore, were promptly commissioned and we joined their team in mid-1973.

The site on the south side of Napier Road slopes steeply to the north. Being an isolated site, an access road was required and this was planned to also serve an adjacent site intended for future use as diplomatic headquarters for another country. Many native trees grow on the site which commands an excellent outlook over the adjoining golf course and botanical gardens.

The brief required large exhibition, library and theatre space to be provided at ground level, so dictating that office accommodation should

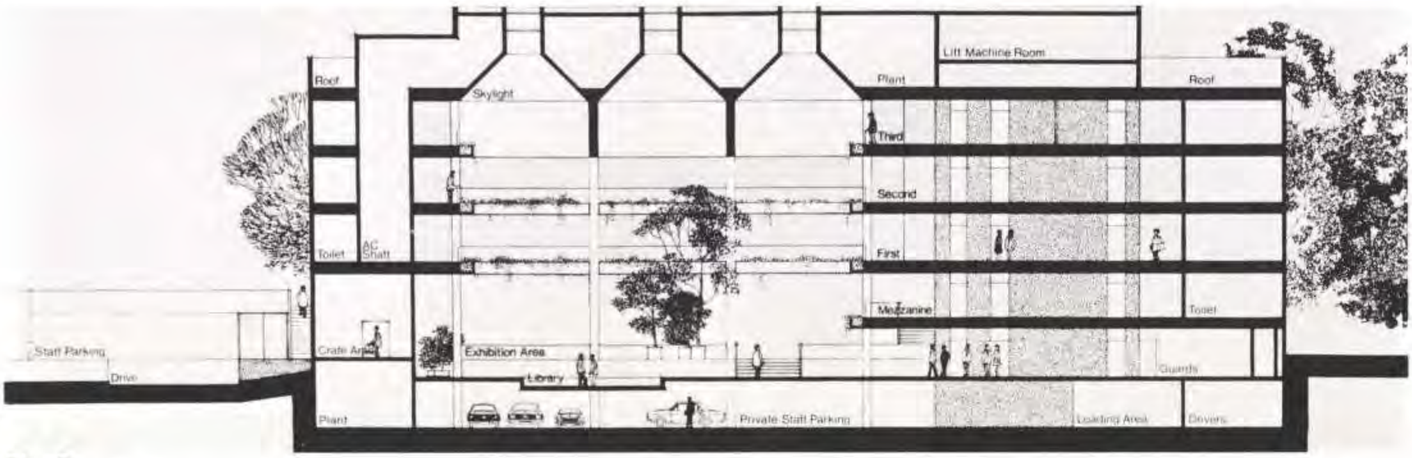


Fig. 6
Singapore High Commission: section

be on the upper levels. The function of such a building may be clearly divided into the following – student examination; trade, general administration; defence and political accommodation. The architect maintained this division by situating each department on a separate floor.

The ground floor was conceived as being the heart of the building and an effort was made to link all the other functions visually to this area. An open space approximately 24 m square and 13 m high forms an internal court surrounded by the upper levels. This space, known as the Great Hall, is covered in by a grid of deep reinforced concrete beams capped by four 'inverted hoppers' which pass through the roof plant room and enable natural light to shine into the space beneath.

Four boreholes were drilled as part of the site investigation. These showed the sub-strata to consist of medium dense clayey silt whose bearing capacity increased only slowly with depth. In addition there was evidence of a high water table. With columns on an 8 m grid their loads at foundation level were substantial and, due to the low allowable bearing pressure, they required pad footings of excessive size. Our choice then lay between piles and a raft. With construction time being important and since the thick raft itself formed an impervious basement slab, we opted for the latter. This was designed as a flat slab with drop panels and included an allowance for an upward water pressure which could conceivably arise should part of the system of relief drains ever become blocked. Naturally the retaining walls were similarly designed to withstand such water pressure. The raft was also designed for relative settlement between the heavily loaded areas and those carrying less load, e.g. under the Great Hall.

The entire structure was designed in in situ reinforced concrete with all external concrete being faced with white ceramic mosaic tiles to provide resistance to tropical mildew. Basically the floors are designed as flat slabs with drop panels but around the Great Hall they span in one direction only. There are many different design load conditions but the basic office areas were designed for a live and partition loading of 4 kN/m^2 plus a load of 0.5 kN/m^2 for ceiling and services.

The east and west elevations at Floors 1 to 3 have 14.5 m spanning edge beams which are themselves supported at the corner of the building by the cantilever portion of the north/south spanning edge beams. The north and south façade above Level 3 steps out from the facade line at the lower levels which means that Level 3 beams carry the loads of both this level and the roof. The change of alignment of the façade necessitated the edge beam load being carried back to the offset column by cantilever brackets. This resulted in a design

Fig. 7
Singapore High Commission under construction in 1976
(Photo: Liang's Photo Service)

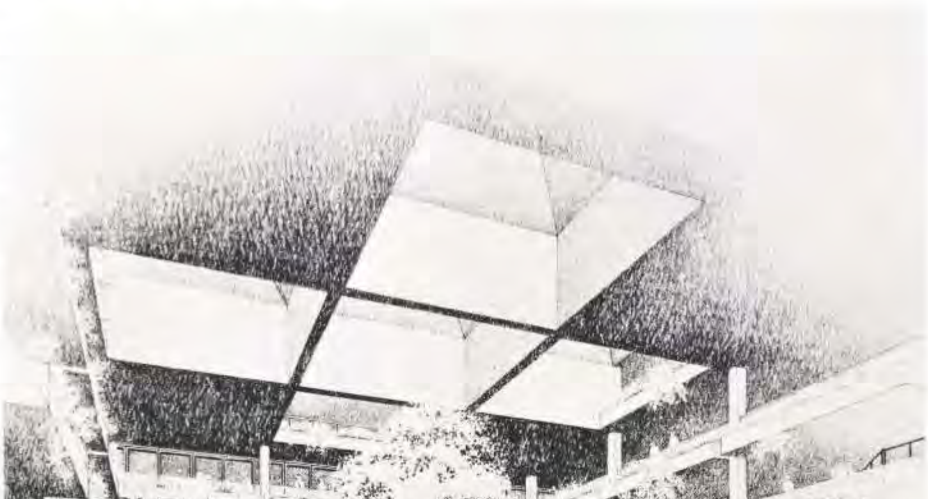


Fig. 8
Singapore High Commission:
The Great Hall

problem involving bi-axial column bending and beam torsion with some complex reinforcement detailing. To make matters worse drain-pipes were required to be cast into certain of the columns involved. However, diligent site supervision enabled the reinforcement to be fixed satisfactorily and concreting proceeded normally.

The deflections of the long-span beams and cantilevers concerned us and we specified cambers in these members. To date their behaviour has been as expected. Nevertheless generous tolerance has been incorporated by the architects into the window fixing.

Singapore Office assisted us with the design of the roads, drains and the ancillary works. This was itself an interesting exercise since substantial stormwater run-off occurs in the catchment and has to be carried via culverts into the existing monsoon drain at Napier Road.

Construction started early in 1975 and the structure was completed approximately 12 months later. It is envisaged that practical completion will be achieved in late 1976.

Credits

Client:
Australian Commonwealth Government

Architect:
Godfrey & Spowers Pty. Ltd.

Main contractor:
Paul Construction Co.

AUSTRALIAN HIGH COMMISSION, KUALA LUMPUR

Shortly after being appointed as engineers for the chancery building in Singapore, we were similarly engaged for the project in Kuala Lumpur. The architects for this were Joyce Nankivell Associates with whom we had worked previously on the design of the grandstand for the Perak Turf Club at Ipoh in Malaysia.

The site of 0.75 ha is situated near other diplomatic missions and within a fast developing area of large scale commercial developments. Although the site was probably rather restricted for the size of building envisaged, approximately 13,940 m^2, the architects attempted to produce a concept which enhances the site, is functionally comfortable and does not attempt to compete with neighbouring buildings in terms of height or bulk.

The final design has produced a structure having one basement level and five floor levels which provide similar facilities to those in the Singapore building. The building is L-shaped with a ground floor reflective pool contained within the re-entrant angle. Vertical lift, stair and services transportation are contained in a large shaft sited at the junction of the two wings of the building and this shaft is continued above roof level to contain cooling towers and lift motor rooms.

Architectural requirements have largely dictated the structural form. With structural steel-

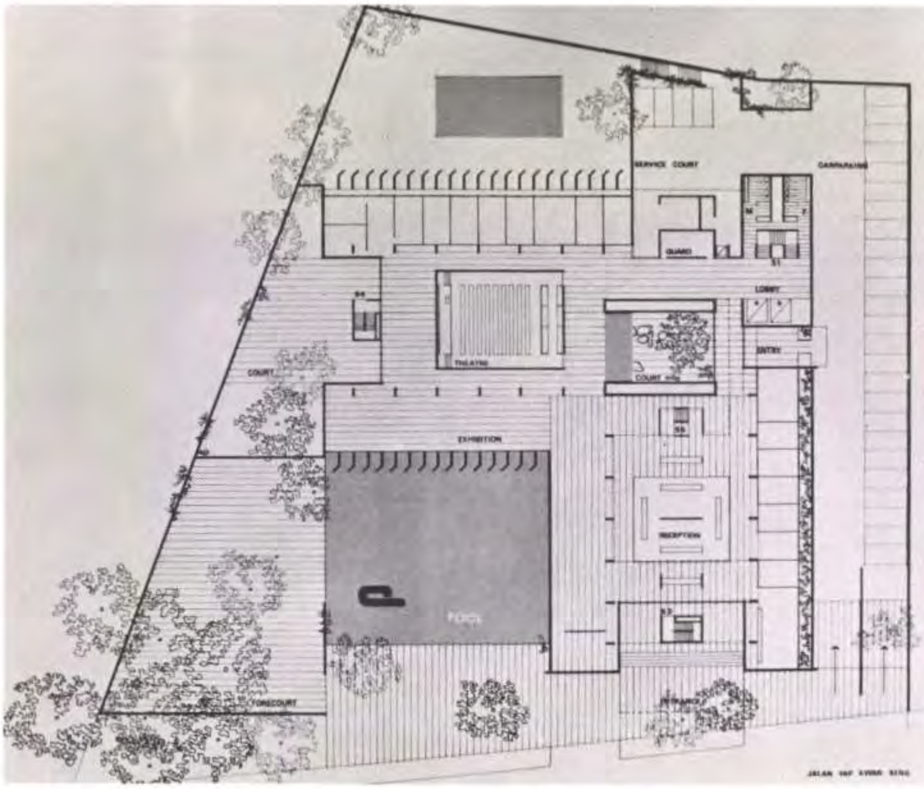
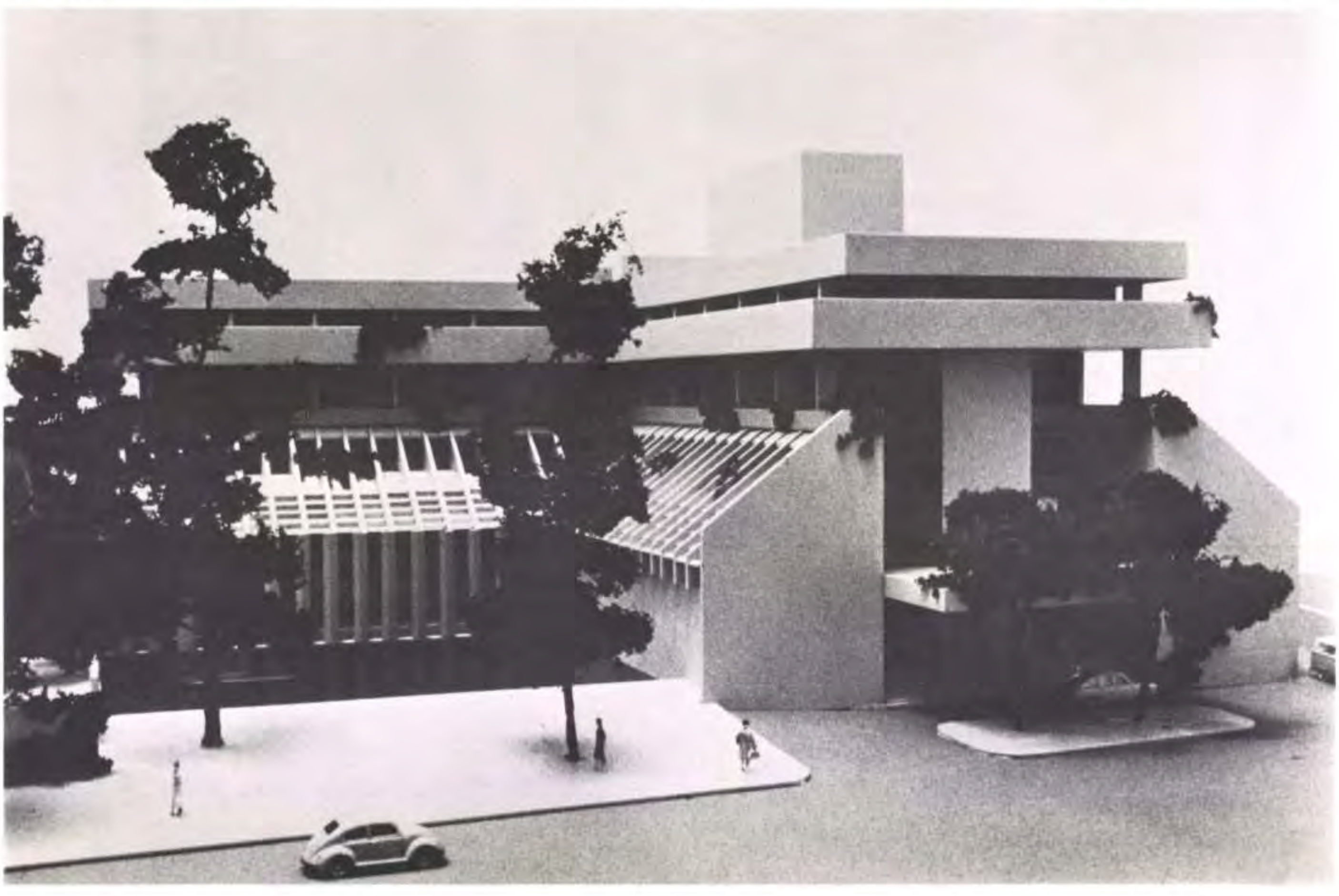


Fig. 9
High Commission, Kuala Lumpur: plan at ground floor

Fig. 10
High Commission, Kuala Lumpur (Photo: John B. Gollings)



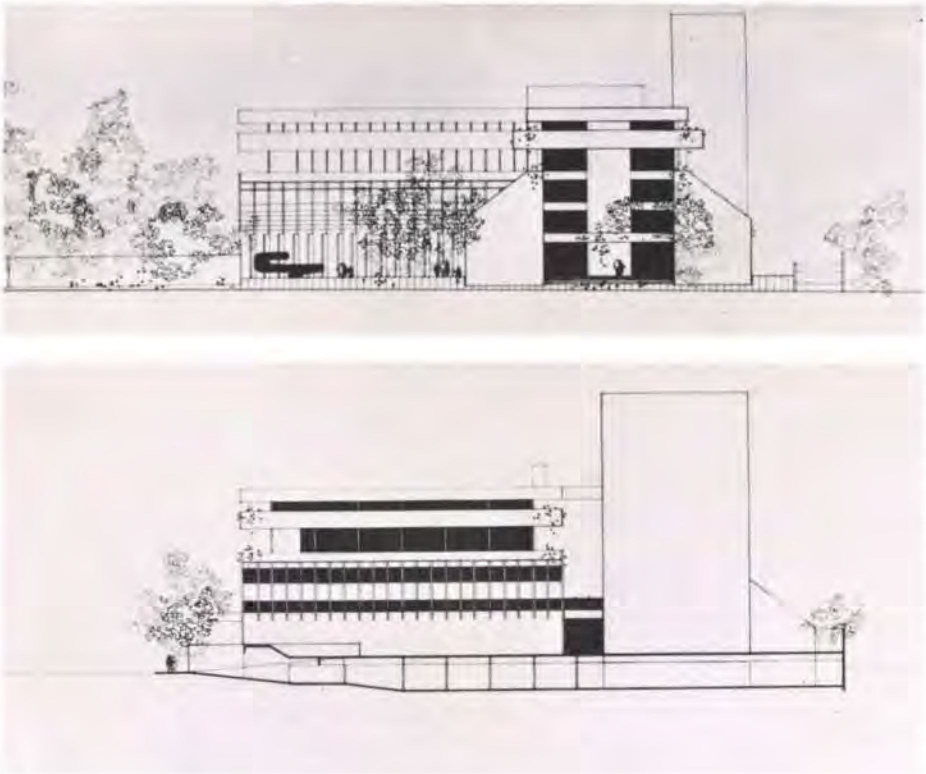


Fig. 11
High Commission, Kuala Lumpur: elevations

work and post-tensioned concrete being rarely used in Kuala Lumpur, the constructional material decided upon was reinforced concrete. The basic structure comprises a series of multi-storey portal frames at 5.1 m centres which are connected together at each level by the 175 mm deep floor slabs. The portal beams span up to 17 m and are perforated at regular points to allow for the passage of services. As the deflection of the portal beams was considered to be critical, the suggested design sizes of beam and perforations were modelled as a Vierendeel girder and run on a computer program in order to estimate as closely as possible the deflection to be expected at mid-span. The deflection was found to be small, due mainly to the high stiffness of the T-beam action from the 175 mm slab. This structure is surrounded (between Ground and Fifth levels) by a series of 'fins' which, in conjunction with horizontal connecting hoods, form an extension to the basic building form. The structure has been planned in precast concrete to enable the builder to take advantage of an apparent constructional time-saving. All external concrete will be covered by Shanghai plaster – an impervious applied plaster – in order to prevent the formation of the tropical mildew. The ground floor slab forms the roof to the underground car park and plant rooms and has been designed as a flat slab with drop panels.

The stability of each wing is afforded by the portals when the wind blows parallel to the main beam span. When the wind blows at right angles to the beam span, stability is achieved by framing action from the edge beam/column combination and (via the floors) from the lift and stair walls.

Different areas of each floor have been designed for loadings likely to be encountered in their future usage. The general office areas have been designed for an imposed load of 6.5 kN/m^2 and the lobbies, corridors and theatre for 5.75 kN/m^2.

The library, filerooms, pool area, plantrooms and the rooms of the secure area on the sixth floor have been designed for higher imposed loads in the range of 7–12 kN/m^2. Masonry partitions have been designed as live loads on the floor.

A site investigation had been carried out in 1969 but related to a building different from the High Commission now under construction. Since the position of boreholes was unsuitable for the current proposal, a supplementary investigation was commissioned.

The strata comprise clay soils of various sand contents which in most cases has been shown to overlie rock. The level of this rock, where proved, varies across the site from 11 m to 19 m and 10 m below ground level. Ground water levels indicated in the original investigation report are shown to vary between 5 m and 10 m below ground level. These readings were checked by piezometer and the maximum water level was found to be just below basement level. On this information, all sumps and pits below basement floor level have been tanked and relief drainage has been provided under the slab. The ground slab is to be protected by a plastic membrane and all construction joints are to be sealed by waterbars. One of the worst floods on record took place approximately two years ago during which water rose to floor level of the adjacent building on the north-east side. The latter is below basement level of the proposed High Commission.

Consolidation tests have been made on samples to check the anticipated settlement of any pad footings used. In addition chemical testing has been carried out to determine the acidity or alkalinity of the soils.

The original site investigation recommended that piled foundations should be used. Local practice in Kuala Lumpur has been to use precast concrete driven piles and two preliminary piles were installed and tested during the design period. One pile was driven to the rock layer where the latter was at the shallower depth. The other pile was driven deeper but not down to the rock. Testing of each pile established behaviour and design criteria for the piles.

The tender documents incorporated precast concrete piles but made allowance for an alternative method to be submitted. The Frankipile method was subsequently proposed and accepted. Piles are driven to tube refusal and with half the piles installed, lengths correspond approximately to the rock depths anticipated.

Preliminary testing of the *Frankipiles* has shown that the settlement at working load in the areas where rock is deepest is 2 mm which is less than the differential settlement allowed in the design. Construction started on site early in 1976 with a contract time expected to be in the order of two years.

Credits

Client:
Australian Commonwealth Government

Architects:
Joyce Nankivell Associates

North Whitfords Urban Development, Perth

Ken Gilbert

Introduction

The North Whitfords Urban Development project is situated 20 km north-west of Perth. The area is 630 ha and comprises part of a three-developer suburb of Whitfords of 1900 ha. It is located immediately adjacent to the Indian Ocean and as such, has the attraction of cooling sea breezes on summer afternoons.

Fig. 1 shows the relationship of the Whitfords area development to the Perth Metropolitan area and to the future urban growth.

House building rates in Perth over the past five years have averaged around 11,000 per year with serviced lot production at 9,500. Prior to then, in the late 1960's, lot production was even further behind the house building rate. To alleviate that problem the West Australian Government offered to enter into an agreement to re-zone the Whitfords area land in return for guaranteed lot production and a maximum lot price. The developers were obliged to pay for all costs involved in providing services to the lots.

Taylor Woodrow Corser Pty. Ltd., a consortium company comprising Taylor Woodrow (Australia) Ltd., and Corser Homes Ltd., a West Australian project house builder, accepted this offer and in May, 1970, the first on-site roadworks of the Whitfords scheme commenced.

Ove Arup & Partners are consulting engineers and principal agents for two of these Whitfords developers – Taylor Woodrow Corser Pty. Ltd. on the North Whitfords project, and Estates Development Co. Pty. Ltd. on the South Whitfords project.

To date, the three developers have created 5,800 serviced lots, 2,400 of these being in the North Whitfords project. Fig. 2 shows the current extent of the North Whitfords project.

The land and housing industry in Perth

Housing in Australian cities for the average family consists of about a 120 m², 3–4 bedroom house, either individually designed, or a project builder's standard model. It is placed on an allotment separately purchased by the home seeker about a year previously. No house design bears any relationship to the next door neighbour's. An Australian desire for individuality in housing to date prevented the success of any attempt to plan a total estate combining land and house.

A typical Perth house would be constructed with concrete slab floor, double brick outer walls, single brick interior walls and clay tile roof for A$20,000.

Fig. 3 shows a typical Whitfords residential street some four years after the initial earthworks commenced.

Perth housing industry consists of builders and land developers. Our commission and involvement in the Whitfords project is to the land development industry.

Obligation of the developers and of the government

Under the agreement with the West Australian Government, the three Whitfords developers were obliged to pay the cost of all the services provided to create the lots. (Before 1970, urban development in Perth had been on a strictly step-by-step basis. The statutory authorities first extended a major service trunk main. All those fortunate enough to be able to discharge sewerage into that extended sewer main or to obtain water by linking to the extended water main could subdivide their land into building allotments.)

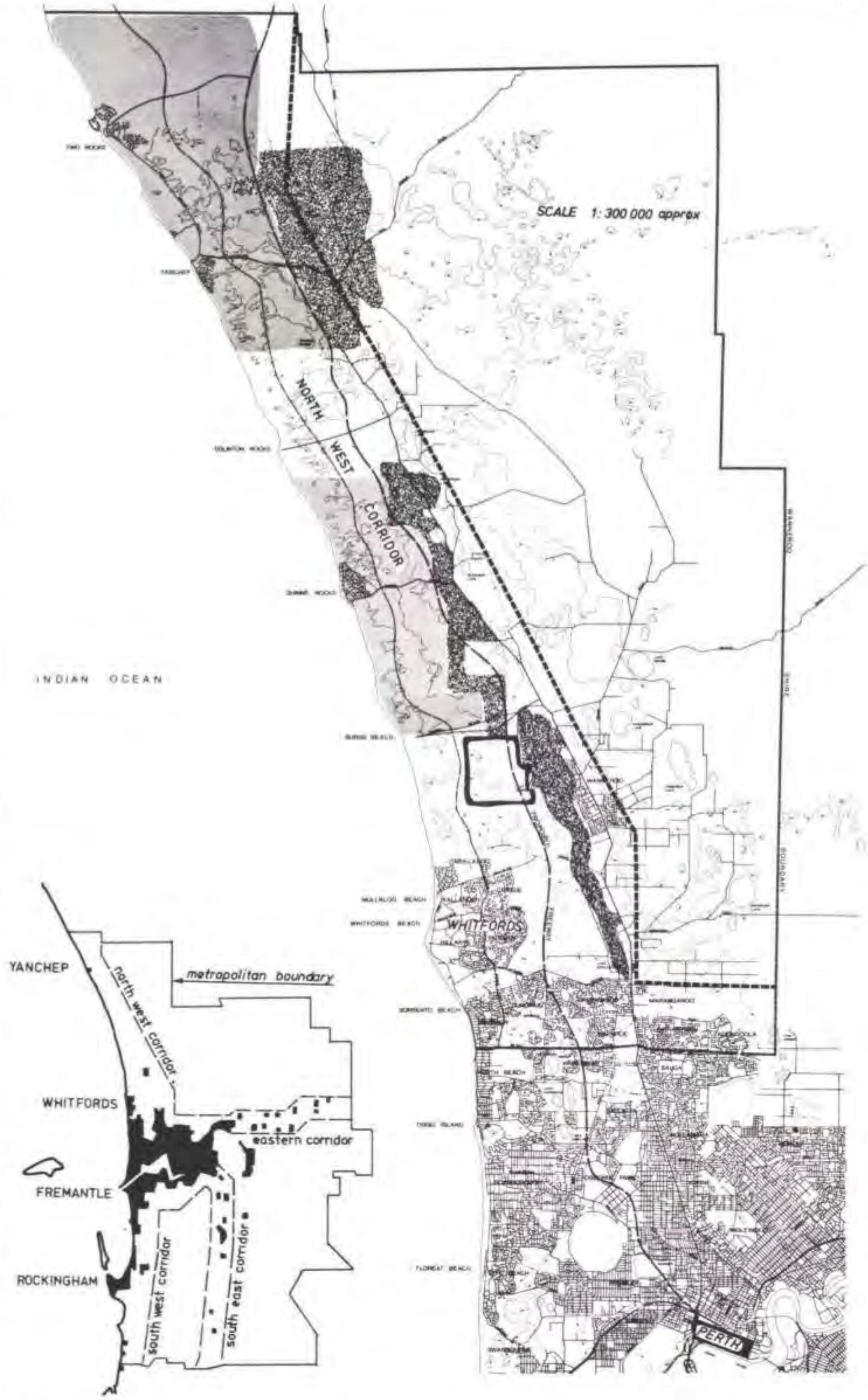


Fig. 1
Whitfords and the Perth Metropolitan area

Fig. 2
Aerial photograph of North Whitfords
(Photo: Associated Surveys)

Fig. 3
A typical suburban street in North Whitfords
(Photo: David Gordon)

Whitfords was created by the developers paying to extend three controlled access highways a total of 16 km, to extend a 600 mm water supply pipe 10 km, and to lay two 600 mm gravity sewer trunk mains for 5 km to a sewerage treatment works.

All internal services were to be provided. Water, gas, electricity to each lot, constructed roads with totally piped stormwater and a sewerage reticulation network to provide for each lot, were required. A footpath network was to be provided and the developer had to contribute, cost free, public open space to the extent of 10% of the total gross area of development. Land had to be provided free of all costs for the construction of schools at the rate of one 3 ha primary school per 800 lots and one 10 ha secondary school per three primary schools. The land required for three controlled access roads to pass through the development had to be given free. The developers were required to produce lots at the rate of at least 800 per year.

Fig. 4
Typical road structure and kerbing
(Photo: David Gordon)

The agreement with the Government limited the sale price of the lots when averaged over all lots to a certain predetermined maximum. This last obligation, however, proved unenforceable with inflation. Currently the lots of the Whitfords development are selling at an average price of A$10,000.

The Government, on its part, agreed to instruct the Metropolitan Regional Planning Authority to re-zone the land from rural to urban.

In retrospect, the developers are of the opinion that this Whitfords agreement advanced development of their land by about five years compared to the older domino type development. It succeeded in obtaining lots without a high raw land component in cost. This saving in the acquisition of the land for development was spent on the extension of trunk services. It is worth noting that this development agreement was the forerunner of many similar deals and most land development in Perth is modelled on the Whitfords project.

Design parameters and standards

Roads

The soil type throughout the whole of the Whitfords project is dune sand with intrusions of an old dune structure calcified to limestone. Consequently, the sub-grade for the road is of a particularly high standard and more importantly, free from strength variations due to moisture changes. When vibrated with a 1.8m steel roller, the sand sub-grade has a California Bearing Ratio of around 11, although the single-size sand particles make it unsuitable for wheel loads directly.

The general pavement material used is 150 mm crushed limestone sub-base with 75 mm graded quarry crushed granite-diorite mix for base primed with 70/30 cut back bitumen and sealed with a 20 mm asphalt hotmix. On bus routes the sub-base is increased to 230 mm thickness. A lay back kerb, 300 mm × 125 mm at the back, is extruded on top of the primed base. This joint between the hotmix and the kerb acts as a stormwater channel. The distance between kerbs is 7.4 m for a normal residential street and 10 m for a bus route.

Fig. 4 shows a typical road structure of crushed limestone, crushed rock base and hotmix seal with the kerbing. Design of road profiles is controlled by a maximum grade of 11% for residential streets and 6% for bus routes. Minimum road grading for stormwater run-off is 0.6%. Earthwork is relatively cheap in sand and general quantities are 12–20 m^3 per metre of road, including road verges (at generally 2%) and grading into the lots (at generally 17%).

We have carried out extensive experimentation in trying to achieve economies of road base construction, particularly since quarry run crushed rock is becoming more expensive. Trials have been carried out in three residential roads on the use of blast-furnace slag and lime as a stabilizing agent for the in situ sand. Unfortunately we have not met with a great deal of success because the stabilized sand tends to develop too much strength and produces a shrinkage crack pattern with too wide a spacing, therefore large cracks.

Limited success has been achieved on the third trial where cracking was deliberately induced at 24 hours after stabilizing with a 4-tonne steel roller with 50 mm angles diagonally welded on the drum. Even then a 50 mm hotmix overlay was needed to prevent the reflection of these cracks to the surface thus offsetting any cost advantage.

As a side experiment, we did find that by mounting an additional spray bar on the stabilizer machine and adding bitumen emulsion to the slag-lime-sand, a reasonably good base resulted. However, instinct is that a result obtained by two opposing mechanisms probably is not a good solution. Additionally, the costs of so many components gets somewhat too high to pursue seriously.

Stormwater collection

The sandy nature of the Whitfords area results in economies of stormwater systems. Collection into piped drains is only necessary for run-off from the road carriageways and the adjacent road verge. Discharge from house yards and roofs is disposed of within the lot boundaries by soakwells directly into the sand. It should be pointed out that in the Whitfords area, as in most of Australia, stormwater drainage is piped separately from sewerage.

The pavement run-off and the pipe sizes are calculated using five-year flood return periods. A Perth five-year, five-minute storm has an intensity of 120 mm/hr.

As outlined above, the kerb acts as a drainage channel which leads to simple grated road gullies covering a 1.05 m diameter precast gully pit. Their capacity is dependent on the slope of the road in which they are installed; on average, in an interceptor situation, they are capable of accepting 0.02 m^3/sec., and in a sag location, 0.035 m^3/sec. In addition to the design of the stormwater reticulation for a one in five-year flood, an assessment is made of the overland route of a one in 100-year flood, and adjustments made to road gradings to ensure property damage cannot occur.

Stormwater disposal

The Whitfords terrain is principally the result of overlaying sand dunes and consequently tends to form a series of natural unlinked catchments. A comprehensive drainage system to the sea is therefore not economic. Use is made of the extremely porous nature of the soil and the piped stormwater is discharged into, in its simplest form, a rectilinear hole in the ground and allowed to soak away. The proven long-term absorption ability of the soil is 0.1 m^3/sec. per 100 m^2. A storage capacity, calculated using a mass diagram, is provided. This generally results in a hole 1.3 m deep.

We have been concerned at the unsightly appearance of the disposal sumps, which must be fenced for safety. Our current experimenting is in two directions. One is to sink a 300 mm diameter bore down to the sub-soil water level and, by packing a bulb at the bottom with crushed rock, attempt to deliver the stormwater run-off directly back to the aquifer. Trial bores

Fig. 5
Pond for stormwater disposal
(Photo: David Gordon)

have achieved disposal capability of up to 1.3 m^3/sec. but the problem is in stilling the water sufficiently at the top of the bore to settle any suspended material. It has been found that the bore base silts up in only one season sufficiently to halve its capacity. Road oils are the principal offender.

The second and more successful approach is the creation of artificial lakes with sufficient surface area to hold peak discharges and a gently sloping edge to allow the immediate lake edge to act as the absorption area.

Such lakes can only be created, of course, where the ground water table is sufficiently close to the surface to allow practical excavation for a lake. An example of the created lake at South Whitford indicating the outfall and the multiple weir siltation trap is shown in Fig. 5.

An interesting by-product of the sandy nature of the soil is the ready availability of subterraining water. It is common for a house lot to have its own bore and to pump from this aquifer for garden watering. Usual construction is to sink 840 mm pipe segments to the water table and mount the pump motor at that level with a 75 mm pipe down to a suitable sand strata with a 4 m long stainless steel screen on the lower end. Generally the economic depth is considered to be 15 m.

Sewer reticulation
It is a condition of the subdivision approvals given for the North Whitfords project that reticulated sewerage be provided to each lot. Generally the topography of the area results in major catchment basins where it is cheaper to construct a pumping station than excavate a deep line through ridges to the next basin. Reticulation sewers are asbestos cement pipes and precast concrete manholes for the first 1 km of run, after which the long-term effect of hydrogen sulphide build-up has been found detrimental to both asbestos and concrete.

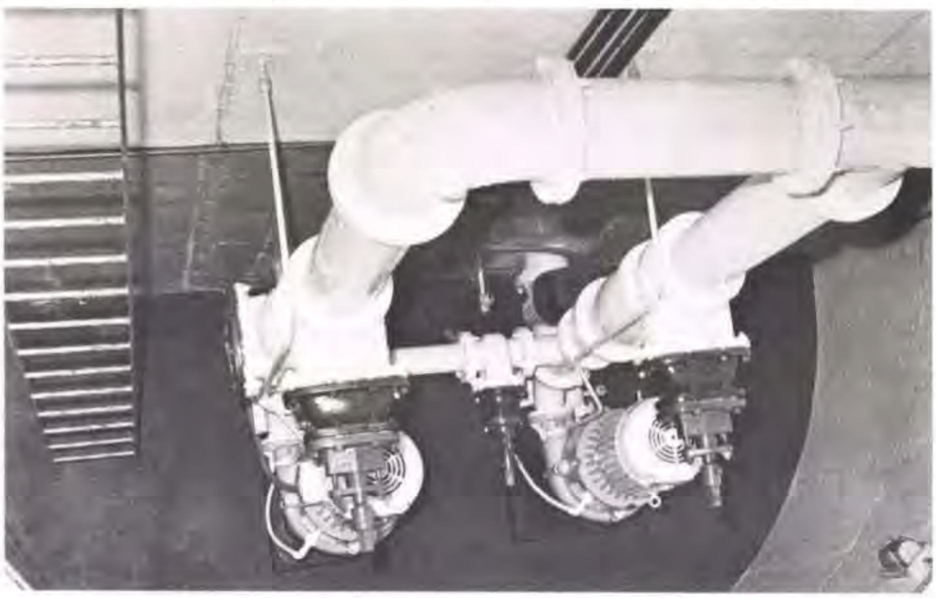


Fig. 6
Sewerage pumping station
(Photo: David Gordon)

From there on glazed vitrified clay pipes and brick manholes are used. The maximum manhole spacing is 90 m and a 1:150 gradient is the minimum for 150 mm pipe which generally is capable of serving up to 400 lots. Sewer design is on the basis of 180 litres/day/person with half the flow considered over eight hours. In the Whitfords area the majority of the sewers are laid above the ground water table so that, in general, no allowance is necessary for filtration. The standard of construction is such that on the few occasions where the sewers are laid under water, infiltration allowance is 50 litres/day/person.

Sewer pump stations
In the North Whitfords project there are eight pumping stations. To date, six of these have been constructed. The largest is a double circle layout with equipment in the centre and the wet well around. The remainder, all smaller and up to 20 litres/sec. are single concrete cylinders, 3.6 m internal diameter, with a central dividing wall between the pump motor sets and the wet wall. Operational and stand-by pumps are provided, both types on automatic level operation, and all electrical equipment is mounted in a pump house above ground. An emergency alarm is provided back to the State sewerage authority depot to indicate pump failures, and an over-flow capacity is provided outside the pump unit area to provide for periods of power failure up to two hours at peak in-flow.

Fig. 6 shows a pump set in one of the stations. The pumping is into concrete pressure pipes with rubber ring joints which discharge into appropriate gravity sewers, thence to trunk mains. Sewerage eventually flows out of the North Whitfords project through a 600 mm plastic lined concrete main to a sewer treatment plant some 2 km inland, constructed by the sewerage authority.

An item which has provoked Ove Arup & Partners' research has been to eliminate pressure loss from hair line cracking in the concrete pressure pipes. It has commonly been found necessary to keep the completed rising full of water for up to 30 days to seal these cracks, through autogeneous healing. We are currently investigating various additives in that water to speed up the processes. The pipe manufacturers have told us they are unable to prevent the cracking.

Electricity
To date, the North Whitfords project has been provided with above ground electricity distribution. Whilst it may be environmentally disturbing, it is provided by the service authority free and recouped from the sale of electricity. Our client is currently considering our recommendation that he pay for undergrounding of these supplies. This will add some 10% to the total servicing costs but it is our belief that the improved appearance will attract a better sales price.

Water supply
In all subdivisions in Perth, water supply construction is undertaken at the developer's expense by the State Authority. Concrete pressure pipes are used for sizes up to 300 mm diameter and spun steel above this size.

Gas supply
The Whitfords area is provided with reticulated natural gas piping to each lot, free to the developer, by the State Energy Commission. The gas comes from a natural gas field near Dongara, some 350 km north of Perth, which has an estimated future life of only five years. By then, it is anticipated that the North-West Shelf offshore gas will be on pipe to Perth. This involves the extension of the pipeline some 2400 km north and some 50 km offshore.

Telecommunications
Telecom of Australia lay PVC conduits for the later cabling of telephone services throughout the subdivision during construction. No cost to the developer is applied.

Fig. 7
Caterpillar 621 elevating scraper at work on road construction (Photo: David Gordon)

Fig. 8
Backcutter JCB 70 on sewer line (Photo: David Gordon)

Fig. 9
For sale at Whitfords (Photo: David Gordon)

Construction machinery

The terrain and the sandy nature of the Whitfords soil result in construction being an earth-moving exercise. With road earthworks averaging around 15 m^3 per metre of road, large equipment is employed.

A typical stage construction would be:

(1) The large trees are removed with a Caterpillar D8 dozer
(2) General scrub is cleared by a Cat 930 loader with a scrub rake in lieu of front bucket.
(3) Topsoil is stripped from all roads, sewer lines, and cut and fill areas, to a depth of 150 mm with a Cat 621 elevating scraper, and stored for later respreading.
(4) Short lead earthworks are completed with a D7 dozer.
(5) Longer haul earthworks are cut to fill completed with the Cat 621 elevating scraper. Fig. 7 shows this in progress.
(6) Sewer lines and stormwater drains are excavated with a JCB 7C. Trench batters generally stand up at 1 in 1 as indicated in Fig. 8. Backfill compaction is generally by a 600 mm plate compactor.
(7) Roads are boxed out using a scraper and trimmed with a Cat 12E grader.
(8) Sub-grade is compacted using a 1.8 m vibrating steel roller towed by a Cat 955.
(9) Limestone sub-base is carted onto site and spread with a Cat 955 track loader and graded up with Cat 12E. Compaction is with a three-wheeled 8-tonne steel roller.
(10) Rock base is carted onto site, spread and trimmed using the Cat 12E grader. Compaction is with a multi-rubber tyred roller.
(11) Gullies are installed on the stormwater lines and manholes; tops on both stormwater and sewers are set to the correct level.
(12) The water bound rock base is primed with 70/30 cut back bitumen applied at 1.0 litres/m^2.
(13) Concrete kerbs are extruded on top of the primed base.
(14) Hotmix surface is applied with a Barber Greene spreader to 20 mm thickness.
(15) Topsoil is removed from the stockpile and respread over the road verge and distributed area to a depth of 150 mm.

Contract management

On the North Whitfords project, Ove Arup and Partners have been given a free hand by the client in the selection of a contractor to carry out the work. At times during the life of the project, the construction facilities of Taylor Woodrow International, an associate company of half of the client, has been used. Negotiated contracts at what we consider to be market rates have been set up.

Contracts are generally extended, varying the unit rates by recognized rise and fall formulae. At regular periods, however, a stage of the development is put out to competitive tendering and all tenderers have an equal opportunity.

Schedule of Rates contracts are usually used giving us the ability to design closely ahead of the contractor. Progress payments are monthly on engineer's certificate. Retention during construction is 10%. This reverts to 2½% at practical completion which is then held for 12 months.

On practical completion of each stage of the project, the works are handed over to the relevant statutory authority or Shire Council. In return for these works being passed for these authorities' care, control and management, the developer receives an agreement for a title to be issued for each lot. On receipt of all the requisite agreements the State Titles Office then creates title to the subdivided allotments. At that time, which is about six months after initial earthworks, the lots can be sold. Fig. 9 shows the product of the work advertised for sale. Apparently just a strip of sand – who can say what has gone into the ground – soon to realize an Australian family's dream!

Credits

Client:
Taylor Woodrow Corser Pty. Ltd.
Principal agents and consulting engineers:
Ove Arup & Partners
Main contractor:
Taylor Woodrow International Pty. Ltd.

Capital Tower

John Nutt
Peter Haworth

Capital Tower is a major office development of 45 levels in the centre of Melbourne. It is now nearing completion and when finished it promises to be one of the finest buildings in the city.

The major city centres of Australia experienced a boom in the construction of office buildings in the early 1970's. Some of these buildings are good – many more are now regarded as mundane by a society whose values have developed quickly. In October 1971, Capital and Counties (Australia) Pty. Ltd. started their largest Australian project, Capital Tower.

The site in Melbourne is situated within the business centre of the city near a busy corner. Nearby are the offices of AMP Society (Australia's largest insurance office), Broken Hill Proprietary (Australia's largest company) and the Shell Organization. It was clearly a prestigious position. The client required a building of international status with good aesthetic and commercial impact and not secondary in any way to the neighbouring buildings.

The Melbourne firm of Godfrey & Spowers Pty. Ltd. were appointed architects and, together with the client, had devised the layout which enables a typical floor to be divided into six possible sub-division arrangements with virtually no corridor around what was to be a central core of a square tower. The dimensions of the core to the façade were dictated by the client based on his experience in rental requirements in Australia and overseas.

The architect considered that the structure should be expressed as part of the architectural form. He wished that the size of the window mullions should be kept to a minimum, and that there should be a minimum of vertical structure at ground level. The solution adopted was to use the central core and four corner columns as supports.

Linking together the corner columns along the sides of the building at Levels 1, 13 and 25 are storey-height, prestressed concrete transfer beams, 5.5 m deep spanning 43 m. These beams coincide with the levels of the plant rooms and support the bank of floors above each beam but below the level of the next higher beam.

The cost of a major building is about 50% of the total development cost of the project to the client after land, rates and taxes, and holding charges are added in. With the structural cost of the order of 30% of the building cost, the range of cost of structural alternatives is likely to be only about 2 or 3% of the building cost or 1 to 1½% of the development cost. Greatest economies will result from reducing the time of building. The whole of the planning was directed towards reducing the construction time of the project. A substantial area of car parking was required, necessitating deep excavation. Rather than penalize the overall project construction time with complex shoring and underpinning problems of the excavation below the tower, the car park was sited below ground to the north of the site with only one basement level below the tower.

Similarly the lift motor rooms were kept within the confines of the core to enable early lift installation such that the builder could use these for vertical transportation during the construction period.

To reduce construction time, it was proposed to construct the main external frame of the tower, beams and corner columns, and the internal core, well ahead of the typical floors. With the completion of the beams at each plant room level, the equipment could be installed and the typical floors above could be erected. In this way work could be carried out on three levels of typical floors simultaneously. The roof level plant room and the installation of the equipment, which normally takes 22 or 23 weeks and cannot be commenced until the highest floor is completed, would be removed completely from the critical path of the programme.

We developed the concept of slipforming the core and corner columns, and at the same time jacking up the main beam formwork supported by Bailey bridge trusses attached to the column slipform, with connecting access bridges back to the central core. As the corner column slipform rose, the reinforcement and post-tensioning cables for the main beams could be fabricated and/or fixed. On arrival at the main beam level the forms would be locked into position, the cables reeved into place, the remainder of reinforcement and formwork fixed and the beams concreted. Floors to the plant rooms were to be formed using structural steelwork members and concrete on a ribbed steel deck. Once the latter was cast, work would then be able to proceed both above and below this level without danger. Materials could be handled to areas below this level using a monorail fixed to the underside of the plantroom floor.

This structural concept was incorporated onto a critical path network which also included the mechanical, electrical and lift installation times plus architectural finishing. The network illustrated that it was possible to erect the building in two years and extra allowances to be made for bad weather and industrial stoppages.

Capital and Counties Pty. Ltd. invited three of Australia's largest building companies to comment on the proposals. Each was paid a fee for this service and was guaranteed to be an eventual tenderer. Each builder was given preliminary drawings and an approximate bill of quantities to enable him to envisage the true extent of the job.

Two of the three builders favoured the general construction system suggested – the third requested that the structure outside the core should be constructed in structural steelwork (which had earlier been investigated and rejected on cost grounds). All builders were

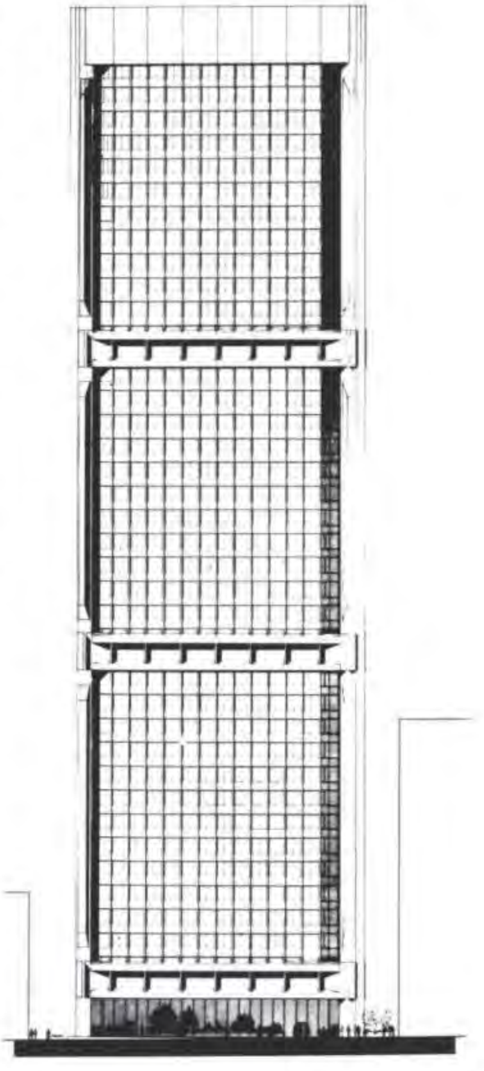


Fig. 1
Bourke Street elevation

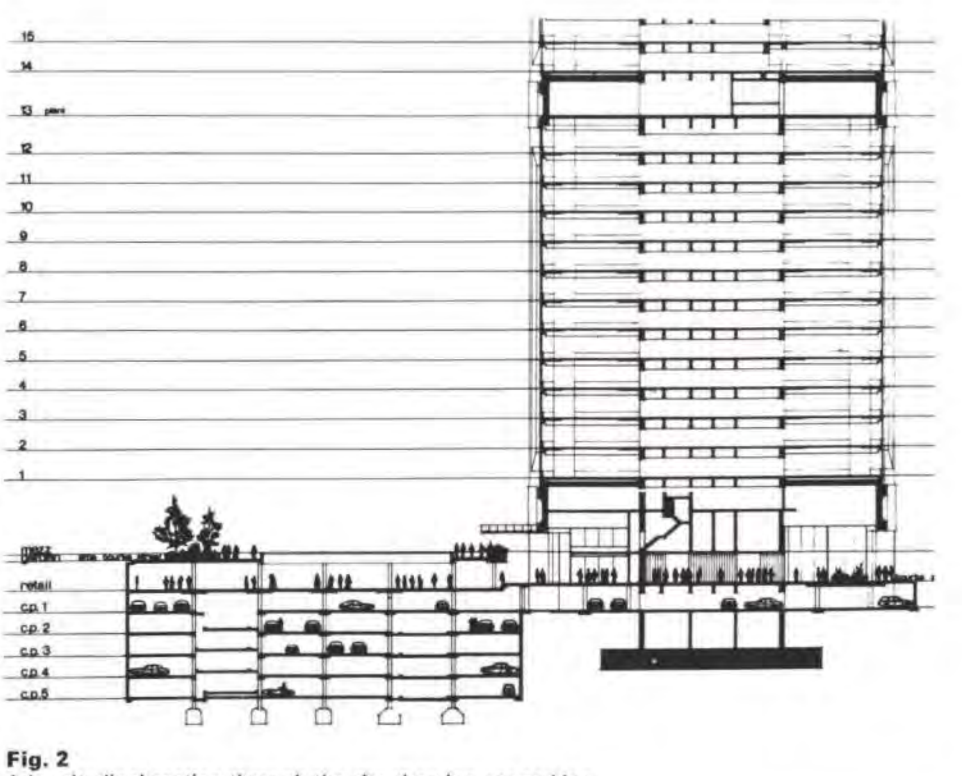


Fig. 2
A longitudinal section through the site showing car parking

Fig. 3
General view

Fig. 4
Capital Tower, March 1976, from the north with the Law Courts in the foreground and the BHP building behind
(Photo: Ian Mckenzie)

opposed to the use of a Bailey bridge as support for the main beam formwork and suggested lighter, specially-made trusses.

The prospect of such a novel job being singled out for industrial action by unions was specifically mentioned by each builder. An allowance of six weeks was suggested for inclusion in the programme to cover this problem. Otherwise the time allocation was generally agreed as being reasonable.

During the subsequent development in the documentation stage, the slipform construction on the corner columns was changed to climbing forms and the temporary Bailey bridge was abandoned to be replaced by steel trusses which would form part of the permanent structure at the roof plantroom level.

Tenders were called from the three builders whose reaction had been earlier sought. The tenders were competitive but 12% above the estimate. Subsequent negotiation reduced the price and it was agreed that the building could be constructed in a conventional manner from the ground up to avoid highlighting the novel construction in what was becoming a deteriorating industrial climate. Costain (Australia) Ltd. were awarded the contract in November 1973.

Unfortunately in the last two years industrial disputes in the building industry have been more prevalent than at any other time in Melbourne's history and Capital Tower has had its share. The internal core has now reached its full height and in June 1976, construction is underway on the Level 26 beams. The time taken to reach this stage is beyond that planned and in retrospect it is unfortunate that the technique for quicker construction was abandoned. Only when strikes and work-to-rule situations are absent do the cycle times for the floors and slip forms equal that planned. A comparison with the OCBC Project, Singapore, conceived and started after Capital Tower, shows what productivity can be achieved in a dispute-free environment.

Foundations

The structural arrangement of the tower resulted in just five distinct points of support; the central core with a total load 480,000 kN and four corner columns with loads 115,000 kN each.

The bedrock throughout the Melbourne area is a Silurian Mudstone, with occasional bands of silt or sandstone, very severely folded,

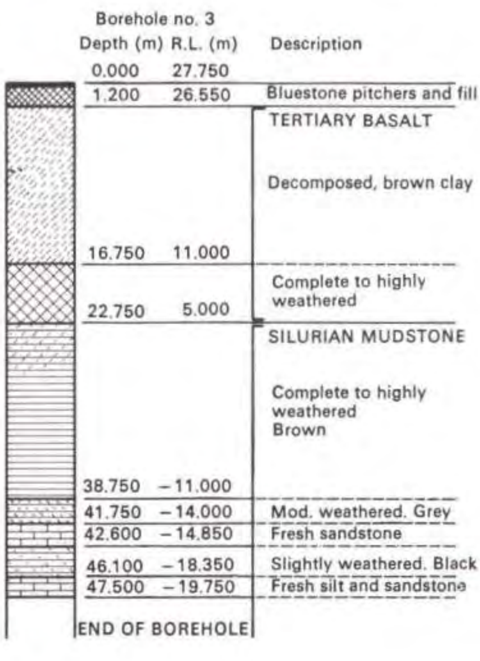


Fig. 5
Soil profile

broken and fractured, with numerous fault zones – both large and small. This formation was weathered to some considerable depth and the surface was showing signs of erosion, when, in Tertiary times, it was covered by very extensive lava flows. The resultant basalt cap has since been weathered and eroded. In some areas it has disappeared completely whilst in others only a thin shield is left, overlain by residual basaltic clay.

The Victoria Uniform Building Regulations stipulate that buildings shall be not more than 40 m high! Such buildings are traditionally founded on the basalt capping or even on the stiff basaltic clay. With the introduction of plot ratio bonuses and extra height allowances for open space at street level, foundation loads increased and designers turned to experience gained from bridge pier foundations for the solution to foundation problems. The solution developed in Melbourne uses large diameter, end-bearing caissons, belled out at a suitable level in the bedrock where this was considered to have adequate strength. Over the years a method was developed whereby the strength of the rock has been correlated with moisture content determinations which can be done very rapidly.

Adopting this system to our particular project produced a lot of piles – particularly below the core – and very long piles, consequently a very expensive solution. We looked at alternatives. Individual pad footings supported on the basalt was ruled out because basalt was not present everywhere and because the corner columns are very close to the site boundaries and centrally loaded pads could not be accommodated.

A fully rafted foundation was closely studied, but was eventually abandoned because rather large differential settlements could occur due to the varying thickness of basalt across the site, which could lead to unacceptable tilt of the whole building.

A raft (or pad footing) for the core only and pile foundation for the corner columns was examined. This proved to be the optimum solution once it was shown that settlement could be kept at a reasonable level. In developing this solution we were advised by Dr David Henkel of our London Office who visited this site, and Dr Jack Morgan, at that time of Melbourne University.

Pile foundation

Design of piles:

We wished to use socket piles to eliminate expensive hand excavations in the bells.

The following was suggested as a design criterion:

$$Q_{ult} = 9\,j\,A_p\,C_p + \sum_0^l s\,\bar{c}\,\Delta l$$

Where Q_{ult}: the ultimate carrying capacity

j: a factor to take into account the reduction in strength due to joints and fissures, $j = 0.5$ was adopted

A_p: the pile cross-sectional area

C_p: the undrained shear strength at the toe of the pile

s: the circumference of the pile

$\bar{c}$: the average undrained shear strength over the length Δl

Δl: finite length fraction of l

l: the total socket length.

Using the already established relationship between shear strength and moisture content, the strength variation with depth could be established by drilling and the necessary length of socket and founding level determined from probe drills on the location of each pile.

Material with an undrained shear strength less than 500 kN/m² was disregarded.

The authorities accepted the general approach but refused to allow combined skin friction and end bearing, consequently we had to calculate our socket lengths on skin friction only, with a safety factor of 2.5. Further, we had to agree to certain allowances for narrow bands of much weaker than average material.

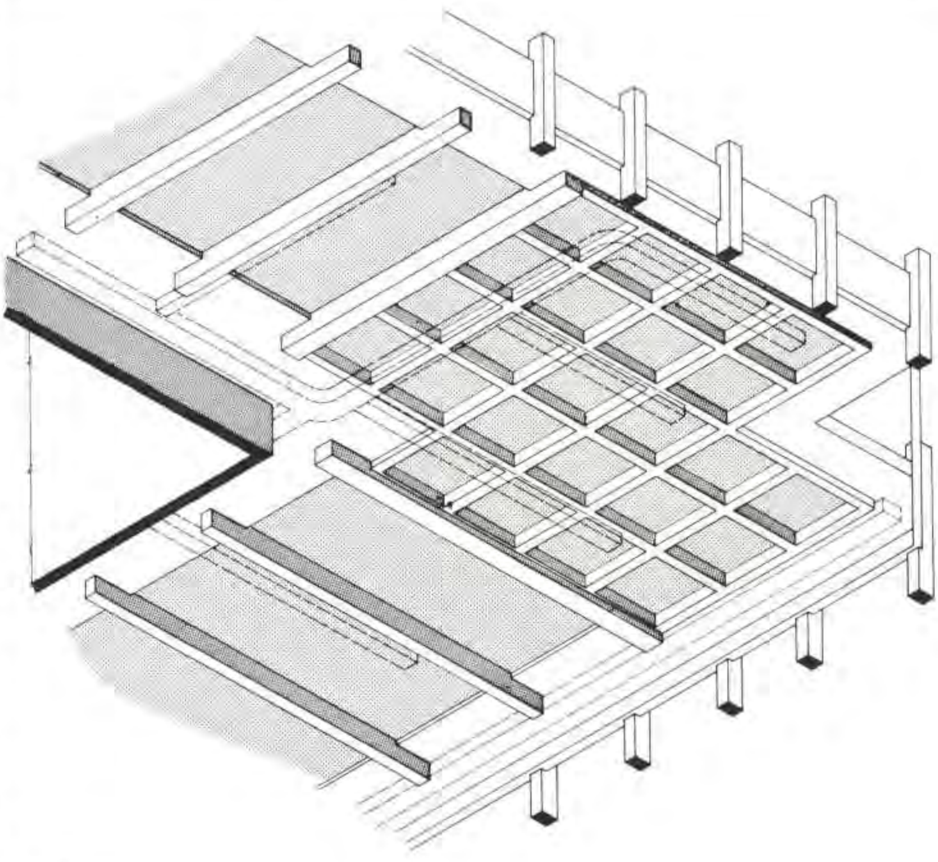


Fig. 6
Typical floor

A construction procedure was agreed with the successful tenderer comprising the drilling of an oversized hole to the top of the basalt, lining as necessary, and hand mining through the hard basalt as required. Below this the completely decomposed and the highly weathered mudstone would be machine-drilled. This length of hole would then be permanently cased. A plug of weak concrete would be cast in the bottom of the hole to seal the casing and the void behind the casing grouted up with a sand/cement grout. It was hoped the pile could be completed dry. When the grout had set, the concrete plug would be broken and the socket machine drilled followed by some final trimming and cleaning out by hand and final inspection.

Our hope that the water flow through the moderately and slightly weathered mudstone would be very low was not fulfilled, and in the end all pile sockets had to be cast under water by the tremmie method. The rest of the pile shaft was completed in the dry. Average pile lengths were 40 m from ground level.

The procedure for predicting pile lengths worked well, but in the last group we struck an igneous intrusion, a completely decomposed soft lamprophyre dyke. We had one site investigation hole in the immediate vicinity plus three drill probes at the actual pile group position and all had missed it. The dyke was about 1.2m wide, dipping very steeply at about 80°. It was in the position of one pile for a depth of about 10 m from the level where we had expected to start the socket.

The pile in question was cased and completed with some difficulties. Although the other three piles in the group were unaffected by the dyke, we could not finish piles within one group at vastly different levels and hence the piles in this group were on average about 8 m longer than estimated from the probes.

The drilling was generally carried out with a Calweld Terradrill 1500. The presence of far more water than anticipated did seriously hamper progress, but, considering the difficulties, the piling operations were completed successfully.

Raft footing

The structural design of the raft slab was a fairly simple operation once the geotechnical part of the problems had been sorted out. The crucial question was whether the cap of basalt was sufficiently thick and strong to distribute the core load to the much weaker underlying residual Silurian clay without over-stressing this material and what settlements we should expect.

The preliminary tests indicated that the likely permissible bearing capacity of the upper layers of decomposed mudstone would be around 300 kN/m². Assuming that the 6 to 7 m thick stiff basalt layer would give a load spread better than normal 2:1, a raft about 30 m square was necessary. The projection beyond the external core wall was then 4.6 m and a depth of 2.4 m was adopted.

The raft was to be founded 10 m below natural ground level. This was the average top level of the basalt layer; the rock surface had to be brought up by about 1.5 m in one corner by mass concrete to achieve this uniform level. The depth of the basalt under the raft varied from about 7 m at the north east corner to in excess of 10 m along the south edge.

The raft was poured in one long operation in late January 1974, and although this is the hot part of the year, no problems were encountered with excessive shrinkage, big heat build-up and cracking. The maximum temperature recorded was 55°C. The amount of concrete poured in the day was 1900 m³.

Settlements

Settlements, and in particular the effects of differential settlements, are important. In predetermining the settlements of the piles we used a three-dimensional stress distribution along the embedded length of the pile and the deformation of the underlying strata.

Neglecting the weaker materials above the level of the socket we considered elastic shortening of the pile shaft. This analysis gave a total settlement of about 33 mm, most of it elastic. There are many reasons to believe that the actual settlement will be nowhere near this figure because of load shedding to the overlying materials. In December 1975 with approximately $\frac{1}{3}$ of the total load applied to the corner columns the measured settlement was only between 3 and 4 mm.

Conventional settlement calculation for the raft, with properties for the basalt derived from a plate-bearing test on an adjacent site, gave a likely total settlement of 55 mm. This too is likely to be a conservative estimate. In December 1975 the measured deflection of the raft was 4.5 mm. Due to construction activities, monitoring of settlements started only when about 10% of the total load had been applied to the raft.

Tower structure

Floors and core

A reduction of the height of the building through minimizing the floor-to-floor height of each storey results in substantial economies in façade, vertical structure and length of vertical service runs. For instance, the reduction of the floor-to-floor height by 300 mm on each floor represents a saving of the equivalent of 5–10% on the cost of the structure. For large spans where the typical floors span between the core and the perimeter columns, we have developed an economical system which relates to the mechanical service runs and the ceiling and lighting system. A beam and slab system is the most economical for such spans. The slab is made as thin as possible, 100 mm fo fire resistance. This then spans as far as possible – 3.05 m in this case – between beams. 580 mm deep beams are required to span the 11.6 m clear distance between the core and the perimeter but where the air conditioning duct runs occur (in a band around the core and around the external walls where a downward perimeter induction system is used), the ribs are curtailed at 1.2 m from the supports and a thicker 230 mm slab is used to transfer the rib shear to the supports. The floor-to-floor height is thus determined by the depth of light fittings under the beams which gives complete flexibility on their positions, and the depth of the air ducts under the edge slabs. In Capital Tower the floor-to-floor height is 3.56 m.

In the corner areas a two-way spanning slab was adopted which utilizes 400 mm deep waffle ribbed slab. The size of the waffle forms used was approximately 1.65 m square – the maximum size of fibreglass former that could be handled by two men. By choosing this size the 100 mm topping was used to best advantage.

The core layout was determined early to suit the planning. The walls were made as thin as possible since this increases the nett rentable area on each floor. The structure was designed around the slipform technique commonly used in Australia. The latter is continuous for a storey-height, after which the form is jacked-off temporarily whilst the reinforcement, blockouts, etc., for the next storey-height are fixed. All reinforcement was arranged to suit this technique and splicing of horizontal bars was planned to enable the core to be slipformed independently in two halves thus allowing continuity of work for each trade.

Transfer beams

The outer frame and centre core combined to carry vertical loads down to the foundations and also to resist the wind forces which act on the building. The wind forces were analyzed by hand, then by computer and finally wind tunnel tests were carried out at Monash University on a dynamic model. It was shown that the frame

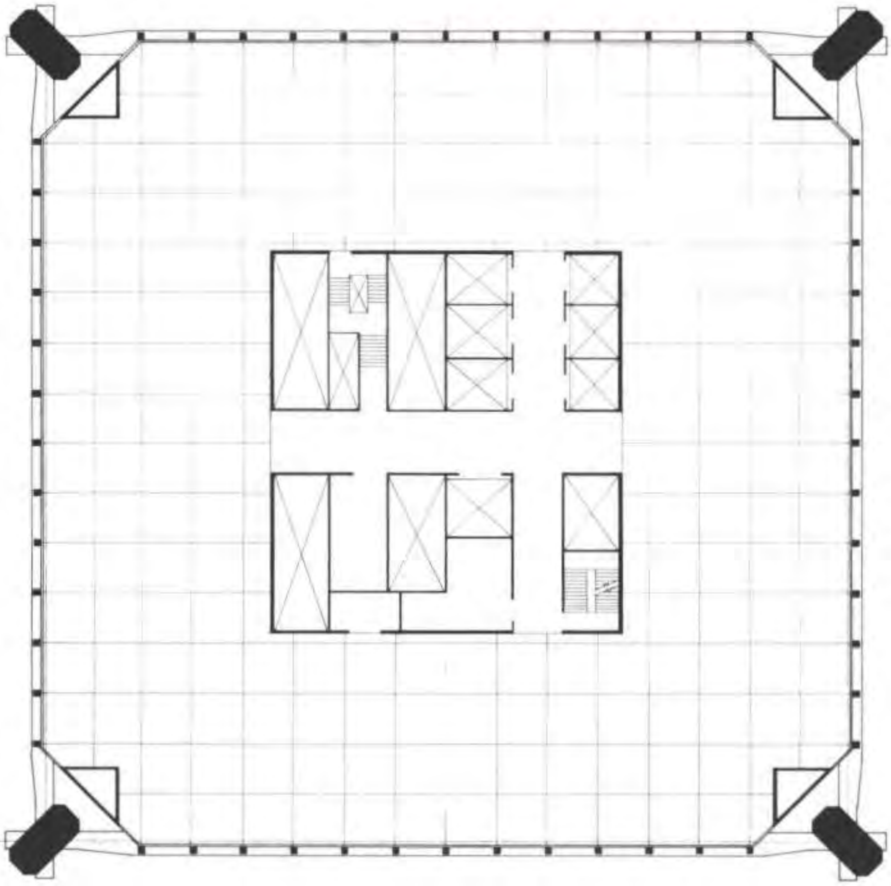


Fig. 7
Low rise floor plan

Fig. 8
Level 1 prestressed beam, west façade
(Photo: Ian Mckenzie)

takes approximately 30% of the wind load and that a transverse pressure of 0.6 W acts simultaneously on the building when the latter is subjected to a wind pressure W blowing on its face.

For the transfer beams, structural steel trusses, plate girders, reinforced concrete beams and post-tensioned concrete beams were investigated during the feasibility stages. Post-tensioned concrete beams were adopted due to cost and depth advantages.

The design of post-tensioned concrete beams in buildings is significantly more complex than in bridges because of the difficulty in allowing for the movement of elastic shortening, shrinkage and creep, temperature effects between outside and inside, and the restraints imposed by the planning.

We started by making the transfer beams and the columns of the external frame integral with each other at their junctions. However, at the Level 1 beams, the shortening movements of the beams caused excessively high moments in the columns and at that level the beams are seated on specially designed spheroidal bearings which allow rotations and movements in all directions. These bearings are located within openings in the columns through which beams pass. These openings and the bearings are then grouted up when the full prestress and loading has taken place and shrinkage and creep movements are greatly reduced. The bearing friction coefficient was measured in test at 0.035. This arrangement is shown diagrammatically.

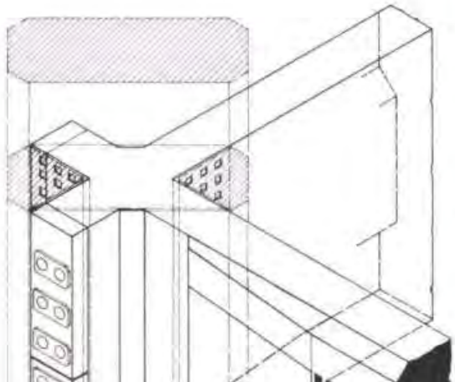


Fig. 9
Beam/column junction: Level 1

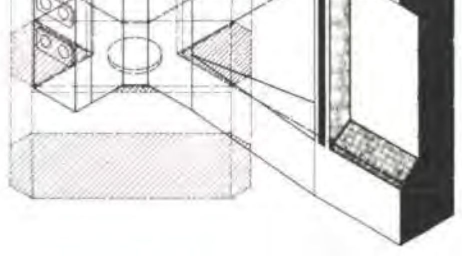


Fig. 10
Beam, mullion and floor junction

At the higher beams, Levels 13 and 26, similar openings were also left in the columns to allow the columns to be constructed above before the beams were poured, but no bearings are installed as the free-standing height of the columns below is great enough to prevent moment overloading due to the beam movement.

The floors at transfer beam levels act as diaphragms stabilizing the beams and providing a connection between the external frame and the internal core in the sharing of the wind forces. Positive connections to the floor are made at the centres of the four beams at each level around the perimeter. To allow the beams to move longitudinally and laterally, the rest of the floor structure is supported on the beams by a series of steel PTFE bearings, bearing friction 0.044.

The centreline of the transfer beams are offset from the centrelines of the mullions because the external face of each must line up to accommodate the window cleaning apparatus.

This imposes a torsion on the beams and a moment on the mullions. This offset was reduced by incorporating a cantilever bracket into the floor above the transfer beams so that the load was applied to the beam near its shear centre. The small mullions are highly stressed and would be incapable of carrying the distributed moment so a pin in the form of a Freyssinet hinge has been introduced at the foot of each mullion. This arrangement of load transfer is illustrated

Fig. 11
Freyssinet hinge at the base of a mullion (Photo: Ian Mckenzie)

The beams have been designed for both working and ultimate conditions.

The elastic stability of the transfer beams was calculated for bending in both vertical and lateral conditions. A simply supported condition was assumed which was conservative for lateral bending. Torsional bending effects were calculated using the theory described by Libby.

The P/δ effects caused by beam rotations were taken into account in these calculations and a factor of safety of about 3 was found to exist under temporary overload conditions.

Torsion is produced in the beams due to a number of causes. Each of the following were investigated individually:

(a) Centre restrained with the ends being twisted due to the vertical bending of the orthogonal beams
(b) Lateral deflection due to shortening of the orthogonal beams
(c) Torsional uniformly distributed load applied along the beam due to eccentric loading of mullions relative to the shear centre
(d) Lateral uniformly distributed load applied along the top of the beam due to friction in the bearings below the mullions
(e) Effects caused by lateral variations in the beam geometry and the cable profiles
(f) Lateral uniformly distributed load applied along the length of the beam due to wind.

A heat transfer analysis was carried out to establish the temperature variations through all members by ambient temperature gradients externally/internally and radiant heat effects. This accounted for ambient temperatures, seasonal variations of the sun's path and daily variations of the surface temperature of members affected by the sun's position. Basic information for this was obtained from the Bureau of Meteorology and analyzed by a heat transfer computer program to determine the temperature distribution through the members.

These temperature gradients try to cause bowing but because of the restraint, forces and stress are developed. They are not linear and even in unrestrained members residual stresses can develop. In addition a lateral bending moment is caused by variations in average beam temperature and major axis moments are induced due to the restraint provided by the foundations to thermal movements of the beams at Level 1. Finally, frame moments are induced due to the differential thermal expansions of the main columns.

Tenders were received from various prestressing contractors and it was eventually decided that the BBR system was most suitable.

The design of the end blocks for the beams was based on the paper by Rhodes & Turner published in the magazine *Concrete* in December 1967. This was based on the theory developed by Zielinski and Rowe. A finite element analysis was carried out to check the effect of interaction of the stresses due to bursting and spalling in the final stressed condition.

We were concerned that good concrete compaction must be achieved immediately behind the bearing plates and that the latter should be accurately positioned. Precast concrete end blocks have been adopted whose sizes have been dictated by the crane capacity – there are two elements per end block.

The cables are fabricated and buttoned on the ground before being placed in the duct and hoisted by crane into position.

The beams at Level 1 were cast in four stages – 2/3 of one beam being cast together with 1/3 of the adjacent beam. Each pour involved the placement of 220 m^3 of concrete transported into position using a combination of two pumps and one crane and kipple.

The corner columns contain a high reinforcement content. They are poured in storey-height lifts about two storeys above the nearest typical floor. The concrete mix included fly-ash in both columns and beams to increase the flow characteristics and reduce the heat of hydration. The required cylinder strength of 48.3 N/mm^2 at 6 months was consistently exceeded by at least 12 N/mm^2.

Curing of the beams involved using the traditional hessian cover and water spray. A temperature check showed that the maximum concrete temperature reached 51 °C at $1\frac{1}{4}$ days after pouring. This contrasted with a temperature of 49 °C reached at $2\frac{1}{2}$ days in pouring one lift of the main columns.

Each beam contains 13 cables comprising 109 7 mm diameter high tensile wires. BBR carried out the stressing procedure as required by the sequence drawing and the cable extensions in almost all cases were close to those anticipated. An 800 tonne jack was used to stress the cables.

Fig. 12
Level 1 prestressed beams from within the ground floor foyer area. Foyer ceiling has not yet been installed (Photo: Ian Mckenzie)

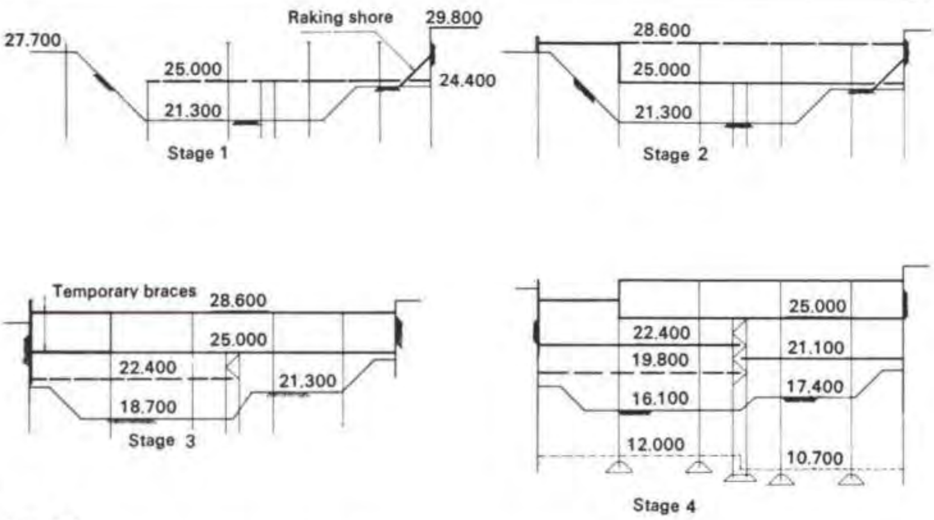


Fig. 13
The car park excavation sequence. Stage 1: perimeter and internal columns installed to full depth and top slab cast. Stages 2, 3 & 4: excavation and installation of slabs and walls progressively downwards

Car park

The car park is a multi-storey, underground structure placed to the north of the office tower. It consists of five split-level parking decks, one level with shops and a restaurant over which there is an open plaza that will be landscaped to create a garden effect. The plan area is approximately 46 m × 40 m and the maximum depth is approximately 21 m below ground level.

The structural system for the slab was chosen to give the minimum overall depth of excavation. Due to services runs, a flat slab with drop panels was most appropriate for the top two slabs. The typical parking slabs below use wide shallow beams 380 × 910 mm, supporting one-way slabs, continuous over six spans. The slabs are 215 mm thick.

The main problem with construction is not, however, the completed work, but the lateral support to the excavated faces, how to excavate the hole, and how to minimize vertical and lateral displacements of surrounding buildings, not least the Law Courts, a building of some historical and architectural merit, situated across a laneway less than 8 m away.

A number of schemes and combinations of different ideas were investigated, the more promising being given below.

(1) Ground anchor support system to the excavation and conventional construction from the bottom up of the reinforced concrete structure. This is undoubtedly the most straight-forward and cheapest engineering solution. Some valid technical reservations can be put forward, the main one being the effect on the surroundings of elastic strain within the earthmass resulting from the removal of 37,000 m^3 of material. However, the main reason for abandoning this scheme was the legal problem in drilling below adjacent buildings.

Eventually a few ground anchors were used but in locations where they extended below roads over their entire length and not under adjacent properties.

(2) The flying shore system was seriously considered, but as a purely temporary structure it was shown to be far too expensive.

(3) To build the central part of the structure in open excavation with batters and then strut against this with shoring when completing the perimeter was a possibility but not economic.

(4) It was decided to construct from the top down by installing the minimum number of vertical supports as steel columns, internally and around the perimeter, and then construct the reinforced concrete structure, excavating under the completed slabs as the works proceeded. Technically the shoring would be the stiffest that could be achieved and the slab at ground level would provide welcome storage space for the contractor on an otherwise very tight mid-city site.

The scheme was slightly modified after discussion with the successful tenderer, and it was decided to install perimeter soldier piles at approximately 2.6 m centres with purpose-made shoring planks inserted between them.

The shoring planks are lattice trusses manufactured from reinforcing rods with a 50 × 230 mm concrete slab as compression flange against the earth face. The tension chord will eventually be concreted into the permanent 230 mm retaining wall and here provide the majority of the horizontal reinforcement required.

The sequence of construction is shown. (See Fig. 13).

Credits

Client:
Capital & Counties (Australia) Pty. Ltd.

Architects:
Godfrey & Spowers Pty. Ltd.

Main contractor:
Costain (Australia) Ltd.

Westmead Hospital Project

Jon Burgmann

The Westmead Hospital Project is a major new health care centre on a 50 ha site in the western suburbs of Sydney about 20 km from the city centre. It is planned as an 885-bed teaching hospital with facilities capable of expansion to 1,100 beds. It will also serve as a general hospital with referral facilities for a large part of the metropolitan region. The complex will include other additional facilities, to be described later.

Westmead is the first teaching hospital in New South Wales to be built from scratch. All others have grown piecemeal, often from colonial origins.

Phase 1 of the project was budgeted at A$120 m. in February 1975 and is the largest job undertaken by the firm in Australia since the Opera House.

Initial ideas and establishment of design team

For many years there have been proposals and the recognition of a need for a major referral hospital (possibly a teaching hospital) in the western suburbs of Sydney. We were first asked to be structural and civil consultants for the Westmead Project about four years ago, but the early years were occupied with planning studies and ancillary buildings.

Initially the hospital architects were interested to explore the possibility of 'interstitial' services spaces; that is, a walk-through services void between user floors. The intention in providing such space is to ensure a high degree of flexibility for services both in initial planning and subsequent modifications.

An interstitial floor also allows future alterations to services at reduced cost and minimum disruption to adjoining areas. Generally this approach leads to long span truss structures utilizing the interstitial floor depth and providing column-free space below for flexible planning. As a corollary the resulting building envelope tends to be a deep plan form rather than narrow blocks. Such facilities have been completed at Greenwich District General Hospital in Great Britain, and at the McMaster Health Sciences Centre in Ontario.

After considerable study and development of alternative schemes it was concluded that the additional initial capital cost for an interstitial scheme could not be justified under present methods of funding. Neither were the large clear spans found necessary when detailed planning was completed.

Up to this period there had been no established brief within which to proceed and the role of the construction authority was not clearly defined. In mid-1974 a new administrative organization was established to oversee policy and planning on behalf of the client, the New South Wales Health Commission. This Project Committee consisted of representatives of the Health Commission, State Treasury, and Sydney University (because of the teaching hospital role).

A Planning and Development Committee was established to provide regular detailed guidance to the architect and consultant team, and also to give specialist health care and technical input. Within this committee there is representation of client, university and planning. The point of communication to the client organization is the project director, a man of broad hospital planning experience specially engaged for this project.

The New South Wales Public Works Department (PWD) is the design and construction authority. The Government Architect's Branch of the Department has supplied a large architectural team and engaged other consultants along with ourselves to assist in design, preparation of necessary documents and supervision of the project. Members of this design team are listed at the end of this article. The design team is headed by a PWD project manager who was formerly the Department's project officer for the Opera House. The project manager has the responsibility of ensuring that design and construction proceed in accordance with programme and budget.

Actual construction work is controlled by the construction manager whose role is described in a later section.

To meet the teaching needs of the academic intake already planned by the University it will be necessary for the hospital to be ready to receive first patients by late 1978. Thus we had just over four years to plan, design, build, equip and staff the first major phase of the project.

As a team we still didn't have a complete brief in mid-1974, but we had a project ahead of us with a time programme defined, and we set to with dedicated enthusiasm. Scheme design was completed in approximately four months, and although it has been modified in the light of more detailed brief information and design development, the concept of the project is basically unaltered.

Content of the project

The main components of the project are seen in Fig. 1 (a). The hospital buildings at the southern end of the site are predominantly

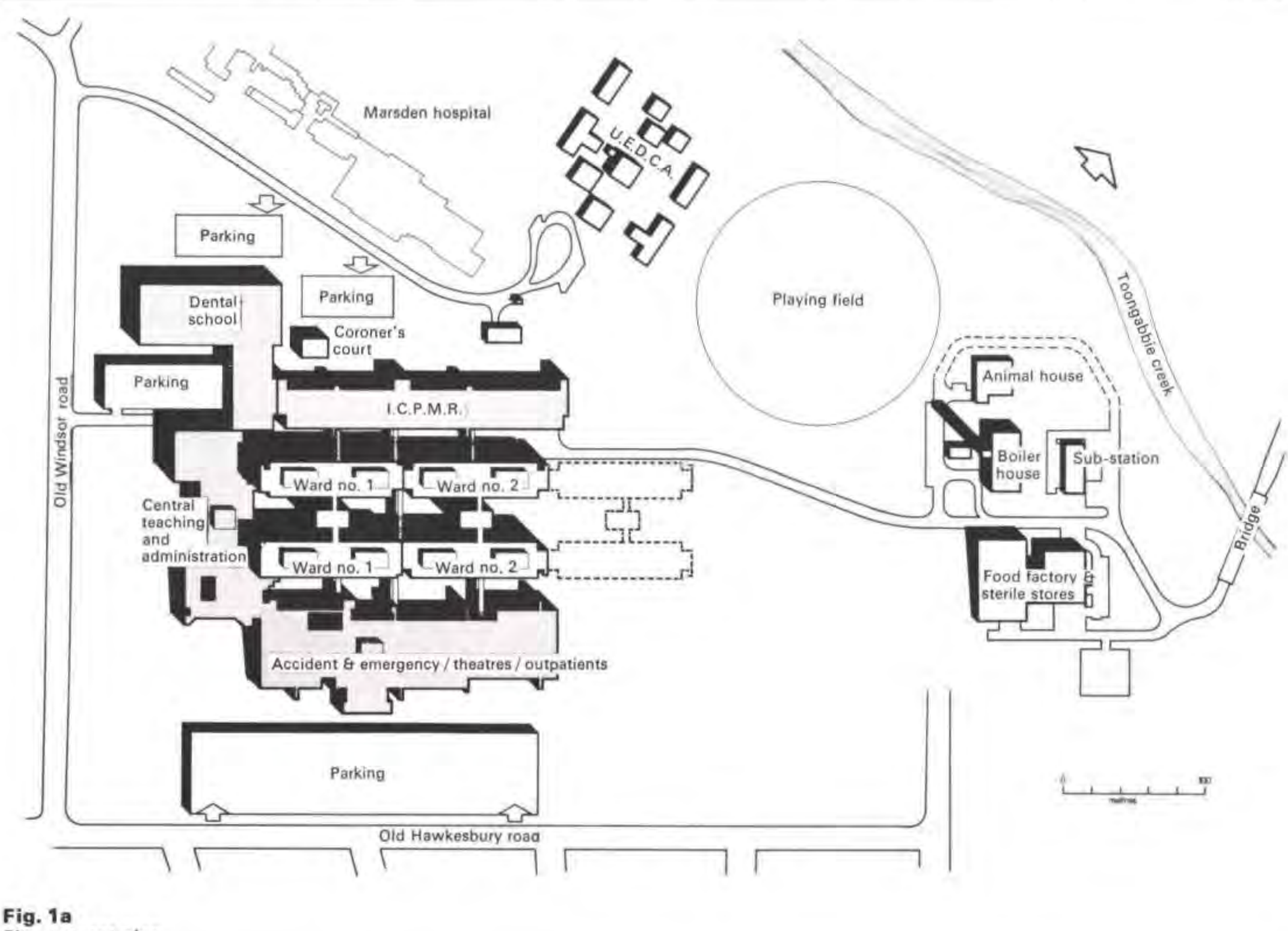


Fig. 1a
Site concept plan

Fig. 1b
Architect's perspective from the south east

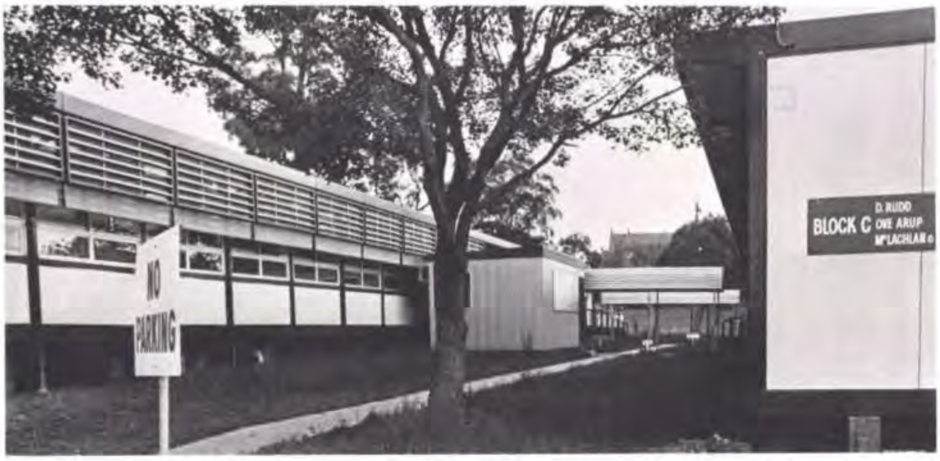


Fig. 2
Site offices for design team at Westmead
(Photo: Lincoln Harvey)

three-storey buildings, with the exception of the wards which are seven and six-storey blocks. Fig. 1 (b) shows an architect's perspective of the main building.

The hospital will provide 885 beds initially, and 1,100 beds if and when the third ward block is built (shown dotted). Ward blocks are H-shaped in plan. A central core in each block serves four ward areas of approximately 30 beds each.

The main treatment block contains Accident and Emergency, Theatres, Medical Imaging and Out-patients.

There is a central teaching and administration block. This contains major lecture theatres, library and teaching facilities because the hospital is 18 km from the university from which students are drawn.

The foregoing facilities are what one would expect to find in any major teaching hospital in varying combinations. However, the plan also provides for a new dental school (to share teaching facilities), and a major research complex in the Institute of Clinical Pathology and Medical Research (ICPMR). The ICPMR is an existing institute which will transfer to the Westmead site in late 1977 with an expanded staff of approximately 400. As its name implies, it essentially conducts medical research and pathology laboratory analysis. In its minor role it will provide pathology services to Westmead Hospital but its major role is as primary referral laboratory for the whole of New South Wales, research institute, and reference laboratory for the World Health Organization in Australia.

On the western side of the site are the existing Marsden Hospital for children and a new Unit for Emotionally Disturbed Children and Adolescents. (UEDCA). These single-storey domestic type buildings have already reached the stage of progressive handover to the client.

The services area at the northern end of the site contains a variety of one and two-storey industrial type structures. There is a zone substation, animal house and central sterile stores department to serve Westmead. The boiler house provides high temperature hot water to the new Westmead Hospital complex, new UEDCA, existing Marsden Hospital, nearby Parramatta Psychiatric Centre and Parramatta Linen Service (hospital laundry staffed by gaol labour).

Fast track design

In the past the New South Wales Public Works Department has almost invariably required complete working drawings for tender documentation, these being produced after approval of scheme design. Frequently the consultants undertake design and documentation only, the supervision and contract administration being performed by the Department's own staff. Thus the conception, evolution and realization of a project consist of a series of discrete stages following sequentially in line.

For some time prior to final scheme design the client and design team had been concerned at the sheer volume of design and construction to be achieved within the programme. It was obvious to all of us that the traditional sequential system of scheme design, detail design, complete documentation, tender, and construction could not be followed if we were to meet our deadlines.

Considerable time can be saved if tendering can take place on scheme drawings with provisional bills of quantities, thus enabling design and documentation to overlap the tender period. Even greater potential time savings are feasible if construction can start before final design and documentation. On the structural engineering side it implies that foundations can start before superstructure is fully defined, designed and detailed. If early planning can provide adequate services zones, then structure may proceed on site while service design and tendering proceeds, rather than wait for complete services layouts to determine such details as penetrations and clearances under beams. The process benefits considerably if a discipline is imposed on the planning so that services/structure/finishes interfaces are predictable.

To establish this discipline repetitive modular grids are used with allocated zones for structure, services, finishes, etc., between defined controlling planes. This design concept and its associated documentation has been evolved by the PWD as a Method Building philosophy.

Thus we have sought as much as possible to overlap all stages of design and documentation, tendering procedures, and construction. Such condensed or telescoped programming has been given various titles but we adopted the American term 'fast track'. It was accepted that inevitably there would be some unsatisfactory decisions made, either in terms of cost or planning. As much as possible we would have to live with these earlier decisions, and in the worst cases there would be some demolition or modification involved. The benefit to be gained was time. It was a necessity to have this time to train medical students and provide much needed health care in the area. It is also true that 'time is money', but this was not the major consideration.

The critics and cynics have equated fast-tracking with back-tracking, but although there have been some errors, it has allowed us to proceed rapidly, where otherwise in all probability we would still be on the drawing board. Fast-tracking has allowed the project to proceed with progressive provisional tendering as soon as enough is known to obtain realistic competitive pricing. To some extent the full benefits have not been attained, because although provisional tenders may go out on time – even more provisional than intended if they are running late – it has not always been possible for the team to produce working drawings in time for actual construction or fabrication.

Nevertheless, at this stage after 12 months of construction work on the main hospital, overall progress is on schedule, key dates have so far been achieved or bettered, and the first intake of patients is scheduled for November 1978.

Expenditure on the project is currently running at approximately A$1.6 m. per month. To meet our programme the cash flow on construction has to increase very shortly to over A$2 m. per month, and at the peak period of construction overall expenditure must exceed A$4 m. per month.

Construction management

Intimately connected with the decision to adopt fast-track procedures was the Department's determination to have a co-operative contractor, on his side so to speak. A selection was made by competitive bids from a small restricted list of major building contractors, and after detailed interviews and negotiations.

Concrete Constructions (NSW) Pty. Ltd. of Sydney were appointed and a management contract agreed which drew heavily on the PWD and quantity surveyor's joint experience on the Opera House.

In a nutshell, Concrete Constructions manage the whole construction, purchasing and fitting out operations on site, and provide input to the tendering and documentation programme. All contracts let are actually sub-contracts to the construction manager, who has one contract with the New South Wales Public Works Department. Thus the construction manager engaged nominated sub-contractors after competitive tendering, usually on a trade basis or limited construction operation. The construction manager makes payment from a secured advance account and does not carry a contractual risk in the conventional sense, but acts as a member of the professional team.

The construction manager is also represented in the design team as a building consultant. In this manner he can influence design and programming to suit site procedures and the prevailing industrial and tendering climate.

An important role of the construction manager, as on any large project, is to ensure good industrial relations. With the current down turn in construction in Australia the industrial climate is relatively stable and calm, but there have been indications that the unions will seek special concessions when conditions are favourable.

The PWD project manager has insisted from the very beginning that the whole design team should be located on site with the construction manager as soon as possible. At a cost of over A$500,000 temporary site offices were provided on site, see Fig. 2. In September 1975 the project manager moved in along with approximately 70 staff from the Government Architect's branch. They were accompanied by ourselves, the services consultant, quantity surveyor, programmer and construction manager.

The experience of being located together on site has been beneficial for communication and progress of the job. However, misunderstandings and poor co-ordination do still occur occasionally, and it is abundantly clear that true team work depends on one's attitude, not where one sits.

However, the achievement to date is considered quite remarkable and the rate of planning, construction and expenditure will set records for this type of project, at least in Australia.

Planning

To return to the description of the main hospital buildings, the planning seeks to provide flexibility for future extensions or alteration without incurring any significant additional capital cost. It can be seen that there is at least one free end to virtually every main block. Considerable effort went into examination of planning modules which were adaptable to alternative uses. A rigid 7.2 m grid was adopted in the north-south direction, whilst sub-multiples were found to be most appropriate east-west. The complex is planned vertically to segregate unlike activities as much as possible.

Level 0 consists of a sub-floor space for services access and walk-through services tunnel distributing major services to all main blocks.

Level 1 is essentially hospital service activities. It contains plant rooms, kitchens, staff facilities and main distribution of stores and equipment.

Level 2 is the public access and circulation level. It also contains teaching facilities, Accident and Emergency and some treatment areas.

Operating theatres and delivery rooms are at Level 3. Most of the in-patient accommodation is in Level 3 and above.

Additional plant is located at roof level of various blocks.

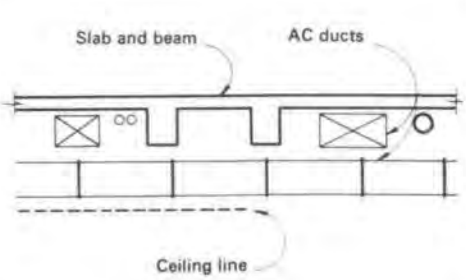


Fig. 3
Relationship of main services to beam and slab floor structure

Structure

It has been often stated, but it nevertheless remains true, that in any hospital building the structure is likely to be the most enduring component.

For Westmead the approach of the design team has been to develop a skeletal structure, 'out of the way', so as to cause minimum interference with planning and services requirements. This ideal must of course be tempered by budget constraints.

Reinforced concrete was selected fairly quickly as the most appropriate material because it readily met the following requirements:

Fire rating
Noise insulation
Vibration and deflection control
Reasonable economy
Technique and resources readily available.

In situ concrete was adopted because it has minimum lead-up time requirements, which is of benefit in a fast-track situation. By the same reasoning amendments to profile or reinforcement can more readily be accepted at a late stage.

A further consideration was the size of the project in relation to industry resources. No single firm would have capacity to meet the required output. In situ concrete enables the demand to be spread more widely across the industry, with separate contracts for each construction operation.

From the above it can be seen that our thoughts about structure were not purely concerned with economy or inter-relation with services, but were also conditioned by the fast-track approach and the practicalities of getting the project built.

As a consequence of rejection of the interstitial services void it was decided to provide vertical services ducts at comparatively close spacing. These coincide with the vertical structure.

A family of structures has been developed which meets the requirements of the ICPMR block, the wards, and the Accident and Emergency/Theatre block. The structural system is used throughout these blocks. The Central Teaching block is more varied, because it contains traditional administrative functions and longer span requirements in the lecture theatres.

The typical family structure consists of twin columns, around a vertical duct, at 7.2 m centres. Twin beams span transversely across the building, supporting a one-way slab nominally 200 mm deep. The twin column/twin beam structure provides for ease of floor penetration, both in the design stage and in the subsequent life of the building. Beam spans vary from 7.2 to 10.8 m depending on function of the block. There is a longitudinal perimeter beam to all blocks, which serves to close the ceiling void and stiffen the slab edge.

The concept is envisaged as a skeleton structure, supporting non-loadbearing façade and partitions, and having defined limits of penetration. Columns and beams constitute the 'skeleton' and are considered inviolate. The slabs are the 'flesh' and are designed as continuous for economy. They function at relatively low stress and are readily penetrated both on a regular modular basis, and in the future if required.

Waffle slab and solid slab floors were considered as alternatives to the beam and slab system. However, for the maximum spans involved, these alternatives were both more expensive and required greater overall depth of ceiling zone.

Fig. 3 shows diagrammatically the relationship between structure and air-conditioning services. Main ducts run longitudinally below the twin beams and the depth between beam soffit and slab soffit is fully utilized for cross-over

Fig. 4
Water blasting in situ concrete to expose aggregate. (Photo: Lincoln Harvey)

Fig. 5
Construction below Level 2, Level 0 slab being formed in the foreground with Level 1 at the rear (Photo: Lincoln Harvey)

ducts and pipes services. Thus the adoption of a beam structure did not add to the overall depth of ceiling zones, this being determined by the dimensions for cross-overs.

A screed has been adopted throughout on all slabs so that step-downs for wet areas can be finalized late in the fast-track programme. Slabs are also designed for heavyweight partitions throughout, because under the fast-track approach it was not possible to obtain an early design decision between the two alternatives. It now seems that a combination of light and heavy partitions will be used throughout.

Externally the exposed elements of the frame are treated by high pressure 'water blasting' to expose the aggregate. This operation can be seen in Fig. 4.

The structural system is quite conventional and reasonably repetitive in design, documentation and construction. We have deliberately tried to keep it that way; again following the fast-track philosophy. The structural performance is well understood and the implications of planning changes or modifications can be quickly evaluated, leaving more time for the analysis and resolution of those tricky problems which inevitably arise.

Reference has already been made to the advantages of predictability in planning, and the structure is perhaps the strongest means of imposing this discipline. The repetition of standard modules and a family of structures enables faster evolution of design, more effective production of working drawings, and faster development of the skills of the operatives on site.

As a team we have found the structure does provide a firm predictable discipline, at reasonable cost. So far it has not appeared to force unacceptable restrictions on planning or services.

A modification of the typical structure has generally been adopted for Level 1. Because there is no requirement for largish spans in the sub-floor space of Level 0, additional supports and piers can be accepted. These have been introduced to reduce beam spans for Level 1 and save on structure costs.

Foundations

The site is bounded on the north-east by Toongabbie Creek and some of the low-lying ground to the north consisted of swamp to a depth of 3 to 4 m.

The remainder of the site was previously orchards, and then a show-ground and race track. During this later phase it was also used as a tip for all sorts of rubbish, including waste from an asbestos cement products factory. Consequently, large areas of site were either swamp or made ground of a most repugnant nature and most unsuitable for direct founding.

Underlying the whole site is a dark grey shale whose weathering has produced the overlying clay soil. Following preliminary and detailed site investigation, and cost studies, it was recommended that all major buildings be founded in the shale using either pad footings or large diameter bored piers (up to 10 m deep), depending on depth below surface.

The recommended bearing capacities given by the soils consultant were based on current recognized good practice in Sydney for this strata. For piers socketed into the fresh shale a minimum of 1.5 times diameter, we were able to use an embedment factor of 3.0. In addition skin friction was allowed for the socket length. The resultant was a net effective bearing pressure of approximately 5.4 N/mm^2.

The first two major hospital buildings to start on site (ICPMR and Ward No. 1) were so located that their foundations were entirely of the bored pier type. All piers above 600 mm diameter were examined by an inspector physically descending the shaft. Check inspections were also made by the soil consultant's geologist. Our foundation design was within budget and in mid-1975 we were embarked on a series of contracts to install ultimately over 1000 bored piers.

Sequence of construction

The sequence of construction has varied between blocks, but it is interesting to

Fig. 6
General view of service area, with the zone substation on the left and the boiler house on the right. (Photo: Lincoln Harvey)

examine the sequence adopted for the first block, ICPMR.

Naturally, foundation piers and pier caps were constructed first. However, because Level 1 was one of the last areas for which the brief could be defined, and also because it would take the longest to design, it was decided to proceed with Level 2 structure and above. When falsework for Level 2 had been stripped, excavation took place for the Level 0 services tunnel between the existing piers. Then the Level 0 slab was formed, followed by Level 1, all of which could be performed under cover of the structure above. The procedure had the further advantage of enabling construction to take place at two levels simultaneously, thus telescoping construction time.

Due to restricted clearance in the sub-floor space below, Level 1 was constructed using metal deck permanent formwork. Fig. 5 shows stages of this work under Level 2 of the ICPMR block. The Level 0 slab for the walk-through services is being placed in the foreground and part of Level 1 is formed in the background. The typical twin columns and twin beam structure can also be clearly seen.

Foundation problems and pier testing

Pier installation appeared to be proceeding satisfactorily, when we were advised that it might well be possible to significantly improve the allowable bearing value of the strata we were encountering. To do so would involve a pier testing programme. After assessment of the potential cost savings it was decided to proceed with the testing. Meanwhile, because of the tight construction programme and our fast-track philosophy, forming of piers continued to the existing design. Six test piers were installed, three each of 450 and 900 mm diameter. One of each was lined the full length to achieve end bearing only, whilst another was lined above the top of the socket and formed with an expanded polystyrene plug at the bottom so that no effective end bearing was achieved, only skin friction over the socket length. The third pier of each set was able to act in combined end bearing and skin friction.

No piers failed in end bearing, and four of them withstood the maximum 4500 kN proof load with almost immeasurable deformation. One of the small diameter piers (450 mm) failed in skin friction as was anticipated, but at a somewhat higher load. However, one of the small diameter piers failed quite dramatically. It settled approximately 30 mm at a comparatively modest load of 180 tonnes, and when the load was reapplied a similar movement was observed.

Naturally we sought to explain this unexpected behaviour and a diamond core was taken from the full shaft length of the pier and 1 m into the shale below. The pier-shale contact seemed satisfactory, but it was noticed that approximately 100 mm core loss occurred in the concrete shaft and foreign matter was trapped in some locations.

Subsequent testing of the concrete core indicated strengths of about 10.4 N/mm^2 at some sections as against 28 to 58 N/mm^2 over the remainder of the shaft. The minimum strength from the core corresponded closely with the level of stress at failure. The occurrence of inclusions and weak concrete coincided approximately with slight pauses in the concreting procedure.

Thus it was concluded that failure was due to defective concrete occasioned by less than satisfactory installation technique. The explanation appeared to be that mud slurry at the base of the pier was trapped in the concrete, due to the small diameter of the pier, congestion of the reinforcement, and the rush of concrete down the shaft after a break in pouring.

There was no evidence of problems with the larger diameter piers, which also had the advantage of being hand-cleaned at the base. Further, it was postulated that there was no trapping action of concrete due to the much greater cross-section; and a modest void or weak patch would constitute a less significant proportion of the cross-section area anyhow.

Thus there was no concern about existing large diameter piers. However, the question remained; what about the 60 or so small diameter piers already installed?

An attempt was made to utilize ultrasonic techniques to detect flaws, but the elastic properties of concrete and the surrounding weathered shale were so similar that results were inconclusive.

Diamond coring was the only readily available method of examining the concrete in the piers, and this unfortunately was both slow and expensive. Nevertheless, a significant proportion of piers were cored over a period of several weeks. Further flaws were detected in about 50 per cent of those examined, ranging from minor core losses, to a major void where the core barrel was actually observed to drop.

It was fortunate that the suspect piers (the small diameter piers) only acted as intermediate support to Level 1. Thus the stability of the main superstructure did not depend on their integrity. Eventually it was accepted that generally there was only a small risk associated with acceptance of the small diameter piers, and this could be covered to the client's satisfaction by guarantees and insurance.

The exception was the Central Energy Plant Area in the ICPMR building, containing the chillers for the whole hospital, standby generators and a sub-station. The vital function of this area convinced us that some precautionary steps were warranted. Therefore supplementary piers (a total of about 20) were installed in that area.

The pier test programme did enable us to adopt higher bearing values for the remaining large diameter piers. These enhanced values were approximately 100 per cent above the earlier recommended values, and the concrete strength of the shaft became the controlling factor.

Due to the construction problems that were highlighted for small diameter piers, no increase in allowable loads was adopted for piers which could not be cleaned out by hand.

Nevertheless the subsequent savings from testing for the larger piers will more than cover the cost of the pier testing. However, whilst we were sorting out the problems of 60 suspect piers which had a building already constructed above, there was a tendency to wish we had never embarked on such an exercise.

Boiler house

One of the first buildings to be constructed as part of the Westmead Hospital Project was a major boiler house. A general view of the services area is seen in Fig. 6 with the zone sub-station in the foreground and boiler house to the right.

The boiler plant (shown in Fig. 7) provides central thermal energy for the existing Parramatta Psychiatric Centre and Marsden Hospital as well as the new teaching hospital and other regional facilities.

There is provision for four boilers, each of 14.6 MW capacity. These will be oil-fired initially, but the boilers are designed for natural gas operation in the future. The boiler plant produces high temperature hot water which is reticulated underground over a total distance of about 1.5 km in pressure pipes. These pipes are supported in reinforced concrete trenches with provision for top access along the whole length.

The boiler house is located on made ground, part of which was previously a swamp. To control differential settlement a piled foundation system was selected. The concrete floor structure is designed for 7.5 kN/m^2 uniform loading, or a point load of 200 kN anywhere. The heavy concentrated load is to allow for replacement and installation of future plant. Superstructure is structural steel supporting lightweight cladding.

Each flue (1.3 m in diameter) requires independent support and provision for thermal expansion. This requirement was met by a lattice enclosing structure of rectangular plan, with an internal cruciform support for flue guides. The structure of square hollow steel sections is seen in Fig. 8. It was assembled at ground level and erected in two major parts. Total height is 45 m above ground.

Commissioning of the first boilers began in February 1976.

Fig. 7
Interior of the boiler house
(Photo: Lincoln Harvey)

Fig. 8
Boiler house flue structure
(Photo: Lincoln Harvey)

Fig. 9
Bridge across Toongabbie Creek
(Photo: Lincoln Harvey)

Site development and access

As outlined earlier, the site had not previously been intensively developed and it was necessary to bring much of the basic services – power, gas, water, sewerage – to site, or at least supplement them. These were handled by various authorities and consultants, including ourselves for stormwater. We have also designed approximately 2 km of roads, about three-quarters of which is complete on site.

It can be seen from the site plan (Fig. 1) that run-off will be greatly increased by the buildings and hard surface areas. In addition, where it had previously been acceptable for run-off to be disposed of by overland flow to Toongabbie Creek, most of the low-lying areas were now to be developed.

Our design criterion was to provide piped stormwater capacity for a 10-year recurrent storm. Above this, run-off would be handled by surface flow on roads and designed floodways leading to the creek. We divided the site into two natural catchments and provided three outfalls to Toongabbie Creek, the largest of which was 1.2 m in diameter.

In practice, it proved difficult for the construction personnel to phase the main stormwater in with roadworks, other services, and access to buildings. In addition there was some doubt about location of future buildings and stormwater design was delayed. Hence, unfortunately, we went through a difficult period early in 1976 when we suffered several months of intermittent cyclonic rains. The stormwater lines were incomplete and many trenches acted as giant moats forming barriers to ready site access. It served to stress the benefit of early establishment of site drainage if at all possible.

As part of the general upgrading of access to the site, one of the earliest contracts was a two-lane bridge across Toongabbie Creek for use by traffic to the services area. In order to clear the 100-year flood level a bridge was required whose total length was 60 m. Geometrically the bridge is very simple with one per cent grade and standard cross fall. The deck is a simple functional form with four equal spans of 15 m. Basically spans consist of longitudinal, precast, pretensioned, inverted T-units at close centres, with an in situ concrete composite deck. Under each footpath a longitudinal beam is omitted to provide a services void. The substructure consists of in situ beam cross-heads carried on circular piers, founded on sandstone which outcrops in the creek bed.

The completed structure can be seen in Fig. 9. The final cost of the bridge, including approach embankments, was approximately A $250,000.

Further improvements to access will be required for the general public, and we have completed traffic studies which highlight those parts of the network which require upgrading. Included in this is replacement of a narrow timber bridge across the railway at nearby Westmead Station.

Situation to date (June 1976)

Bulk earthworks:
started in November 1974, completed sequentially

Boiler house:
started October 1974, now operational

UEDCA:
started November 1974, first blocks being handed over progressively

ICPMR:
foundations started May 1975, superstructure now complete, services and finishes in progress

Ward No. 1:
foundations started August 1975, superstructure now to Level 6, services started

Accident & Emergency/Theatres:
foundations started December 1975, superstructure to Level 3

Central Teaching:
foundations started May 1976, superstructure due to start.

Credits

Client:
New South Wales Health Commission

Design and construction authority:
New South Wales Public Works Department

Architect:
Government Architect's branch, Public Works Department

Civil, structural and traffic engineering:
Ove Arup & Partners

Services consultant:
D. Rudd & Partners

Quantity surveyor:
Rider Hunt & Partners

Programmer:
McLachlan Group

Soils consultant:
Coffey & Hollingsworth

Construction manager:
Concrete Constructions (NSW) Pty. Ltd.

The Kapaluk story : A timber development study in New Guinea

John Nutt

Introduction

Papua New Guinea is now a sovereign nation having become independent in September 1975 after an association with Australia dating from 1883. Fearing the intentions of Germany in the region, the then Premier of Queensland, in defiance of instructions from the Colonial Office in London, sent the Police Magistrate from Thursday Island to take possession of the south-east of New Guinea (Papua) in the name of Queen Victoria. By this act it was hoped that Britain, presented with a *fait accompli,* would ratify the action. Britain decided against it. Germany, meanwhile, annexed the north-east of New Guinea and the Bismarck Archipelago and the next year, forced to act, Britain officially declared Papua a protectorate.

The division of regions between the European colonial powers was the pattern of the time throughout Africa, South America and the Pacific. Some consolidation followed during World War I with the surrender of the German administration in New Guinea to the Australian forces and the establishment of a League of Nations Mandate held by Australia later. In World War II it was the scene of some of the most intense fighting and after 1945 Australia was granted a United Nations Trusteeship over the old German territory. However, the trust territory and Papua have been administered as one country for the past 30 years.

The country has a small population and is very underdeveloped. The road infrastructure is rudimentary which is hampering progress, but there are considerable natural resources, one of which is timber.

Our involvement with the region dates from 1966 when we designed several wharves in the British Solomon Islands. We later designed some town services for a timber establishment and following this, advised the New Guinea Administration on the future town services and transport requirement for Lae, the second largest town. More recently we have established our office in Port Moresby managed by Colin Mathison.

Tropical rain forests

Tropical rain forests cover much of the land of Papua New Guinea. They form one part of a band extending 10 degrees north and south of the equator which stretches through Amazonia, the Congo and the Malay Archipelago. Smaller areas extend into other parts of Africa, Madagascar, India, Philippines, Australia and and Central America. Once almost continuous, huge expanses of this belt have been replaced in the last two decades by plantations of oil palm, cocoa, rubber, bananas and timber crops. For the countries within these confines, the tropical rain forest constitutes a significant national resource when utilized in conjunction with the development of agriculture. It produces sawn timber of great variety and wood chips which are the raw pulping materials for paper.

The tropical rain forests of New Guinea, as those elswhere, are characterized by great richness of species. A two ha sample will contain between 100 and 200 species of tree a foot or more in diameter whereas temperate forests would contain only 10 to 20 species. Alfred Wallace, one of the early naturalists, wrote of the tropical forest in 1878 'If the traveller notices a particular species of tree and wishes to find more like it, he may often turn his eyes in vain in every direction. Trees of varied forms, dimensions and colour are around him but he rarely sees any of them repeated. He may at length meet with a second specimen half a mile off or may fail altogether'. In size the trees do not rival the Redwood or the Australian Eucalyptus and large trees are much less common than smaller ones.

However, there are major species characteristic of each area and generally no more than 20 species are found to comprise the greater proportion of the total volume.

The great variety of timber creates marketing and production problems which the development of wood chip industry overcomes. Recent research and pilot scale studies by government organizations such as the Australian Commonwealth Scientific and Industrial Research Organization have confirmed that satisfactory pulp can be made by the bleached kraft or neutral sulphite process from the mixed forest species of Papua New Guinea timber, and identified the limits of certain species in the mix. In areas where clear filling must be practised to release suitable areas for agriculture, the harvesting of a wide range of species and sizes does not create major problems provided the pulp wood supply remains uniform over a reasonable period. As a result there has been a change in the type of material used for pulping by the paper mills of South East Asia. For example, in 1950 no hardwood was used as pulpwood in Japan, but in 1970 60 per cent of the total was hardwood.

Kapaluk timber lease

Since the mid-1960's, the Papua New Guinea Government has been encouraging the utilization of the forest resources by allocating large leases to groups who would undertake the development of integrated timber industries. Areas of between 50,000 and 250,000 ha have been put out to tender at Madang, Vanimo, Sagari, Open Bay, Kapura, Kamusi; saw mill facilities have been established on several, and a wood chip facility is operating at Madang. Apart from the returns by way of royalties and taxation, the Government expects to take up a substantial proportion of the equity of the development companies. Japanese, American, South East Asian and Australian interests are

Fig. 1
A view beneath the canopy of the tropical rain forest at Kapaluk (Photo: Ken Groves)

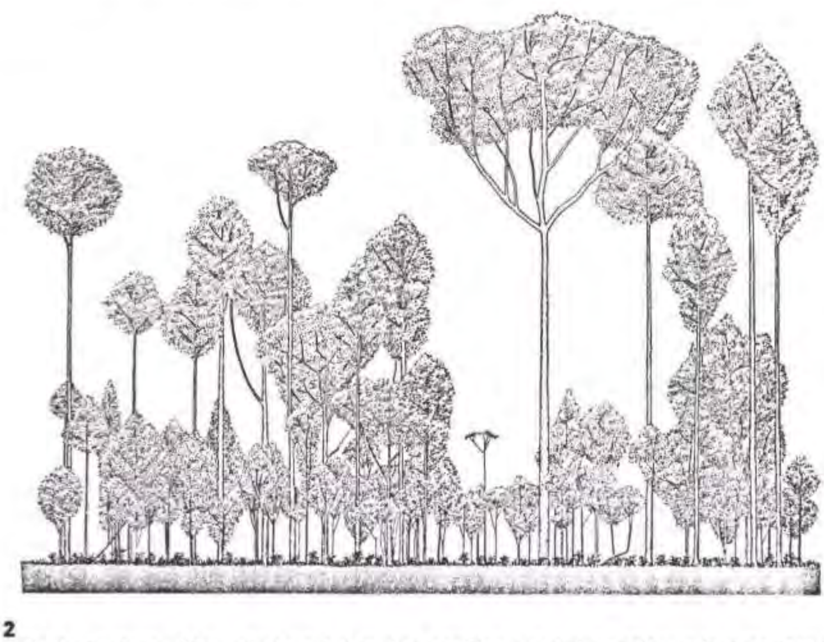


Fig. 2
The vertical structure of the rain forest showing positions and sizes of trees in a narrow strip

Fig. 3
A fishing craft of the Kombi Tribe (Photo: Ken Gilbert)

involved. The emphasis has been placed on local processing, reforestation and the development of business opportunities for the local people in the allocation of these timber rights.

The Kapaluk Forestry Area of 180,000 ha is located on the north coast of the large island of New Britain, west of the Willaumez Peninsula. Apart from a copra plantation at Linga Linga and a small Seventh Day Adventist Mission Station at Silavuti, no development has taken place within the area. The indigenous population is small, about 3,000 from the Kombi Tribe whose villages are on 20 or so islands just off the coast. They live extensively on fish and the product of their native gardens on the mainland. The nearest township of any size is at Kimbe, about 40 km from the centre of the lease area.

Planning

Komatsu Ltd. of Tokyo expressed interest in the timber development lease in March 1974 and after consideration of ourselves and an American consultant, we were appointed for the feasibility study in May. Originally our brief was to investigate the infrastructure and civil engineering component of the development, but during negotiations in following weeks this was enlarged to include the forestry component – timber resources, logging, mill facilities and so on, and operational plans, company structure and staffing.

A programme was drawn up which required the preparation of the report for Tokyo in early August and the submission of the tender at the end of October. We proposed an initial visit to Papua New Guinea for discussions with the Government in May, a field and site investigation in June/July and two subsequent visits for discussions after the finalization of our report.

We brought in a number of specialist consultants to assist us in the study. Keith Gray, a timber resources consultant, and Ken Groves, a logging consultant, were seconded from Forestry Technical Services Pty. Ltd. of Canberra; John Lynton-Evans of Agro Management Consultants of Birmingham, England acted as the processing plant adviser, and Alec McLachlan of McLachlan and Crowe, Sydney was our economist. From Ove Arup and Partners, John Nutt was the project manager, Ken Gilbert the project engineer, Pat Parlour the mechanical and electrical engineer and Brian Blayden, the civil engineer for the field work assistance. In addition, Nigel Oran of the University of Papua New Guinea was commissioned to assess the sociological impact of the development on the Kombi people and Russell Taylor to prepare preliminary township proposals.

Our first visit to Papua New Guinea in connection with this project was made in May 1974 when John Nutt and Keith Gray met Kentaro Yamaza, project manager for Komatsu in Port Moresby for discussions with the Government. During this time we gathered as much on the timber resources and physical characteristics of the local area as was available. This was extremely limited. The only detailed maps of the area were the one inch to the mile series produced during war-time by the United States Department of War, naturally without ground control since the area was under Japanese occupation at the time they were flown. These showed 100 foot formlines but there were inaccuracies in the mapping. However, in 1972 the area had been flown and there was an almost complete cover at 1 : 30,000. These aerial photographs were used as the basis of our terrain evaluation and timber assessment, route location and preliminary township selection.

The coastline is studded with reefs and other offshore navigational hazards and the relevant section of the Admiralty Chart for the area was reported inaccurate. The volume of the Pacific Islands Pilot recommends shipping to stand well clear of the coast because of its hazardous nature.

The tender documents provided by the Department of Forests included an assessment of the timber resources based upon 32 strip counts undertaken by the Department at selected points where circular plots of 20 m radius were located at 100 m intervals along strips 3 km long. However, only trees with a girth breast height above buttress (GAB) in excess of 1.5 m were recorded. These trees however, provided only about 50 per cent of the potential timber volumes and trees in the range 0.6 m – 1.5 m GAB are suitable for chipwood.

The information available on all aspects was clearly limited.

We extended our visit to Rabaul and discussed the lease at some length with the regional forestry officer there who has walked and flown the area several times. We also met some of the local small shipowners who ply the coast in small work and cargo boats, and over the aerial photographs we established

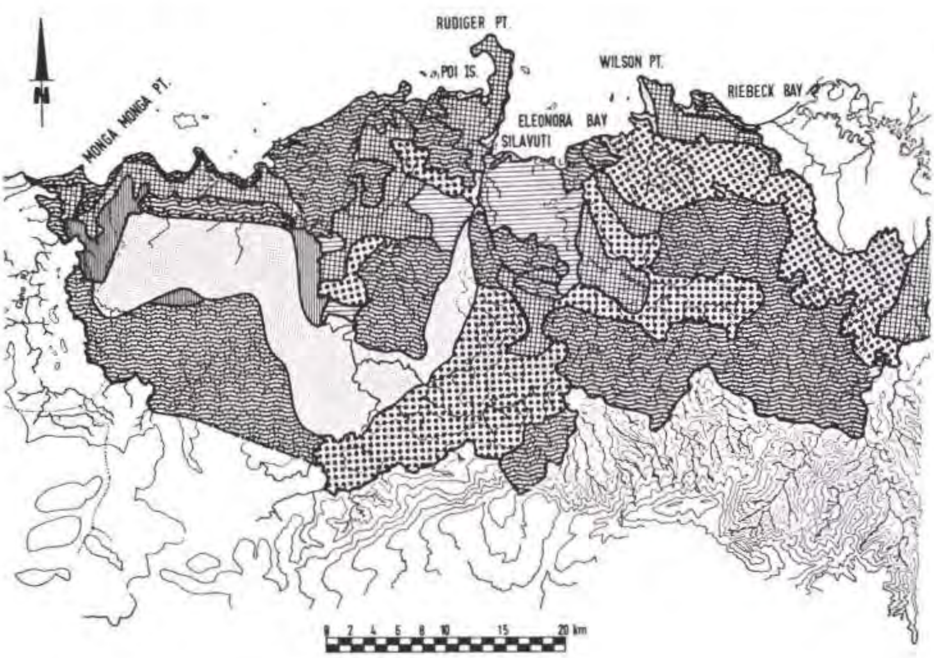


Fig. 4
A terrain evaluation map for logging purposes

Fig. 5
The Kapaluk Timber Lease (Photo: Ken Gilbert)

possible shipping lanes and potential harbour sites.

On the basis of this we planned our field investigation. Having defined the areas we wanted the field team to visit and how we could do spot checks of the timber, we than made arrangements to charter a 60 foot trawler and two work boats, the charter of a five-seater Piper Aztec aircraft to fly in to Linga Linga Plantation, and a four-seater Jet Ranger helicopter to give us access to areas inland.

Before going back we prepared detailed briefs for each of the project team members, finalized the equipping of the field party and pre-arranged the transportation to Papua New Guinea of the personnel from Australia, Japan and England. We went into the site three weeks later with a party of two engineers and two foresters from our team and five Japanese from Komatsu, together with a guide from the Forestry Department.

Field investigation

The area of the Kapaluk Timber lease extends from a precipitous north facing escarpment of the Whiteman Range, whose summit ridge reaches an elevation of 1800 m, to the coast and comprises mountain slopes, flat coastal plains and swamps. The Gaho, Via, Kapaluk and Kulu Rivers can be traversed by small boat but are not navigable because of the difficult entries to the sea. The rivers and coastline are fringed with swamps of which the largest is the crocodile-infested Lake Nhamo.

On site the party split into two groups with the first examining selected points along the coast for a suitable site for a township and processing headquarters. Before we arrived we had done formline plots of these areas and these were checked on the ground. At the most favourable, we then did traverses out from the shore with echosounder and leadline to position the wharf and harbour facilities. We flew the bay in the early morning when the water is very still and reefs below the surface can be clearly seen, supplementing the hydrographic information we had. We then determined the shipping approaches.

The second party struck inland. The foresters spot-checked the tree counts we had available and supplemented those with counts of the smaller sized trees not recorded previously. The engineer and logging consultant examined the ground conditions, looked at potential bridge sites and sources of material and planned the overall methods of logging and roading. They then took a workboat up the Via River and looked at conditions in the rougher limestone country toward the interior. Later the helicopter came in and flew the team to other areas inaccessible from both coast and river.

When the team came out of the jungle it was joined in Port Moresby by the processing plant specialist, and the economist. Over the next 10 days the complete operational plan was worked up in preliminary form. By the time the team dispersed to their various offices, there was an agreed framework within which to complete the details of the study.

The facilities

The key of the study was the determination of the most economical product mix. We looked at log exports, sawn timber, veneer and plywood production, particle board and wood chips. The principal gerera comprising over 50 per cent of the total volume – Homalium, Pometia, Dillenia, Calophyllum and Spondias were all species not widely known on world markets.

The volume of timber at 100 m³/ha was low compared to other tropical forests in the Philippines and Kalamantan. In addition, logging conditions were fairly difficult over a good proportion of the lease area. It was determined that a clear felling operation was required with the establishment of a sawmill and a woodchip facility.

In the early stages of the development until these were operating and the necessary infrastructure established to support them, log exports would provide a cash flow for the project. Veneer production was examined and costed and made subject to a later review, and particle board production appeared not viable on the information available. However, something like 70 per cent of the total volume of the plant output after allowing for defects and waste would have been wood chips.

There were a number of alternative sites to be looked at to use as the centre of operations. Kimbe, the nearest town with a small population and its established infrastructure had obvious advantages, but the longer haulage and roading led to its elimination. A suitable site was located within the lease area which gave adequate shelter and shipping approaches for wood chip carriers of 30–40,000 Dead Weight Tonnage and cargo vessels of 6–9,000 DWT. The ground conditions comprised silty sand, overlying buried coral reefs along the shoreline. Account was taken of seismic and tsunami effects, tide condition, the direction of the prevailing winds and the possibilities of flooding.

Logging

The logging methods proposed were a combination of the client's requirements utilizing his equipment and that dictated by the terrain. Crawler tractors were proposed for breaking out and bunching, and with much of the loggable area on slopes of greater than 15 degrees, skidding was to be by crawler tractor with rubber-tyred trailing arches.

The aerial photographs and our ground inspection allowed us to prepare a terrain evaluation map and delineate the sloping areas and the wet and dry weather loggable areas. Logging units comprising chainsaw teams, bunching and skidding tractors and front end loader were deployed to yield some

Fig. 6
Access to the Via River
(Photo: Ken Groves)

Fig. 7
Helicopter used for access
(Photo: Ken Gilbert)

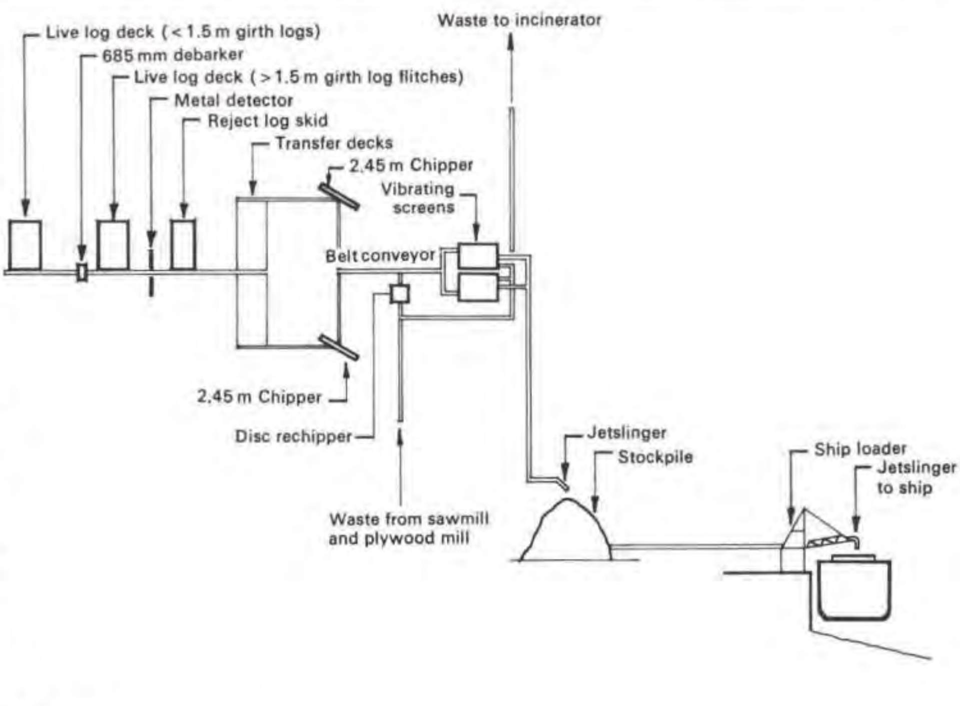


Fig. 8
Schematic layout of chipmill

300–400 m^3 of logs per day per team to the landing stages. A programme, equipment, and manpower schedule was prepared for the first five years of operation.

Road construction was proposed in the year ahead of logging operations in any particular area. Primary roads were to be all weather and be of high standard since a year-round operation at the mill was required to be fed with logs, and where secondary roads were to be used during the wet weather, their standard was upgraded with increased culverts, larger table drains and thicker pavements.

The positions of river crossings delineated the road pattern and it was intended that the major east-west road link would form part of the permanent highway along the length of New Britain.

Logyard

The annual forest turnout was determined at 500,000 m^3 of logs per year and the mill facilities were planned on this basis. Unloading and primary sorting had to be mechanical to accept the volumes, with a lorry to be unloaded at the rate of one every five minutes. After passing over a weighbridge, the lorries would be unloaded by pushing sideways with a front end loader directly onto a live log deck, the logs cut to length and sorted into chip, saw and veneer logs and coded before storing in a logyard.

From the logyards, the logs would be directed to the chipmill, the sawmill or if installed, the veneer mill. Logs have to be debarked for woodchips and a debarker was planned in both mills.

Chipmill

It was proposed that the smaller logs would be taken direct to the chipmill and loaded onto a live log deck to be fed by log chain through a mechanical debarker and metal detector. Rejected logs were to be kicked out onto a skid ramp and the good logs transferred by log haul to a transfer deck to be fed into either of the other two main disc chippers. The output chips would fall under gravity onto a belt conveyor, then to vibrating screens. Reject chips and sawmill offcuts would be fed through a rechipper and recycled. The screened chips would be transported by belt conveyor to be stockpiled by jet slinger. Reclaiming would be by dozer and belt conveyor to the shiploader which would have a luffing and slewing facility for effective ship loading. A jet slinger fitted with a telescopic chute would be incorporated to give maximum chip compaction in the hold. A layout of the chipmill is shown in Fig. 8.

Sawmill

The sawmill was designed to give a large output of sawn timber in the form of sleepers and various board sizes, with a capacity for reclaiming offcuts. This was to be achieved by a duplication of machinery and a considerable flexibility of the flow patterns which the slabs could follow. The basic mill machinery comprised three bandmill breakdown saws, two secondary breakdown saws, a multiple edger and two band resaws for the reclamation of offcuts. As a significant proportion of the sawmill waste – some 20 per cent of input – was to be reclaimed for woodchips a large debarker was located in the sawmill complex. Sawn timber was collected by a chain and docked by multiple saw trimmer into the required lengths. The finished lumber then passed through a sapstain spraying unit and by conveyor to a rotary sorting table to be sorted and stacked for airdrying in sheds.

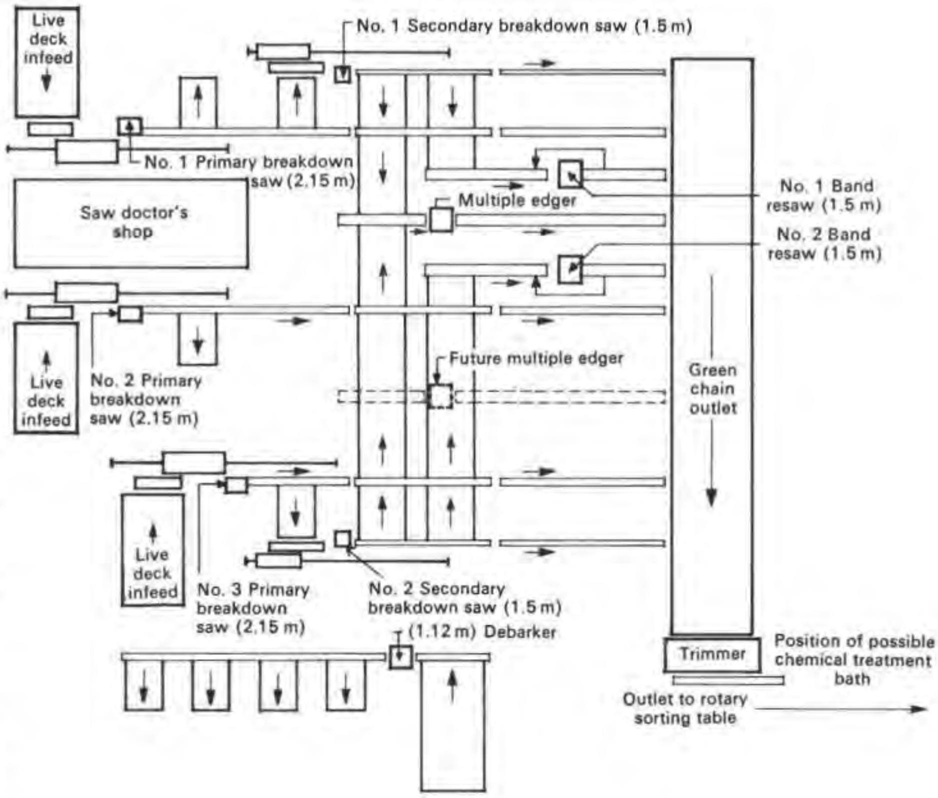


Fig. 9
Schematic layout of sawmill

Infrastructure

The mill facilities had to be supported by their own self-contained facilities. A power station to generate 6 MW, workshops, vehicle maintenance, stores, fuel storage tanks and port facilities were to be provided.

Shipping was proposed for woodchips by specialized bulk carrier either owned or under long-term charter by the operating company and would be fully utilized making 13 round trips to Japan each year. Other timber products would be shipped on scheduled freight services.

An operation of the scale envisaged required an expected workforce of approximately 1000. A township facility was expected to develop as a district centre and was planned accordingly with living accommodation in barracks and married quarters, offices, hospital, school and recreational facility and town services. An airstrip was required for access and radio for telephone and telex communications.

Staffing

We prepared, in addition, a programme for recruitment and training of expatriate and local staff and determined potential conditions of employment after discussions with company and government departments.

In a completely undeveloped area such as this with a limited local workforce and potential problems of land tenure, the ramifications of the establishment of a large complex has to be examined at an early stage. We commissioned a sociological report from a specialist with detailed local knowledge of the area and its peoples and took heed of the advice given.

Considerable social problems have since developed on neighbouring Bougainville Island where Bougainville Copper has established one of the world's largest copper mines and is now faced with secessionist movements which have resulted in interruptions and disturbances.

The policy of reforestation and subsequent agricultural development was to have been the subject of further negotiations with the Government. We outlined a possible programme and established indicative costs.

The timber and pulp wood industries are particularly susceptible to fluctuations of supply and demand on the world market. At the present time the downturn in the world economy has resulted in a postponement of many new projects and the development of the Kapaluk Timber Lease is affected by these world-wide trends.

Credits

Client:
Komatsu Ltd., Tokyo

History

Michael Lewis

I have been asked to write a history of our Australian activities, but to talk of history when looking back over 13 years is either unlucky or unrealistic, so I will do my best to tell something of the story from 1963 to 1976.

For the first five years the Opera House transcended everything (Fig. 1). Our design office was located on the site in temporary offices adjoining the architects and quantity surveyors, overlooking the harbour and the massive substructure of the podium already nearly completed as part of Stage 1. The contractors and our own resident engineering team were located within the podium in temporary offices separated from us by a few hundred yards. On the fringe of the Botanical Gardens, in a hot, sunny climate, it is hard to imagine more agreeable working conditions, but the all-consuming monster, the Opera House, prevented us from enjoying the location to the full. The old Man-O-War Quay was ideal for fishing at lunchtime, but there were only a few attempts and none successful – our time was taken up with a full plate of technical and human problems.

Fig. 1
Sydney Opera House
(Photo: Ove Arup & Partners)

Fig. 2
Prudential Assurance Building
(Photo: Harry Sowden)

Very soon after our arrival in Australia we undertook additional commissions and leased office space in the nearby ICI building to set up a separate team for that work. We were faced with an unusual problem – our reputation for innovative design was rarely questioned, the Opera House was well known and spectacular enough to convince even the most sceptical of clients, but it was generally believed that we were unsuited to 'ordinary' engineering problems. The only way of convincing potential clients to the contrary was to be seen to be doing the simplest and humblest of projects. Although some of these projects were uninspiring, they provided a basis for understanding the Australian building scene and it was not long before we had established a reputation as alert structural designers.

Work on the seemingly interminable problems of the Opera House resulted in a sort of fatigue and many engineers were given opportunities for a change of scenery by working short spells in the ICI Office. These jobs were often of short duration between design and construction, providing a welcome change to the despondency created by design work on a building so large and fraught with problems that one could not see an end to it.

Two early projects which, contrary to the trend, took a long time to go into final design and construction were the Prudential Assurance head office building and Chatswood Station project (job numbers A106 and A108 respectively – starting at A101). Both were large and interesting and being appointed early in 1964 gave us great hope that, with patience, we could establish a worthwhile practice in Australia. The Prudential Building is a central city tower block of 25 storeys (Fig. 2). These and many other projects helped in the juggling which was necessary to keep talented engineers within the firm in Australia as the work on the Opera House receded.

Fig. 3
Perak Grandstand, Ipoh, Malaysia
Client: Perak Turf Club
Architect: Joyce Nankivell

Fig. 4
Allendale Square, Perth
Client: Allendale Holdings Pty. Ltd.
Architect: Cameron Chisholm and Nicol
(Photo: Fritz Kos)

Fig. 5
Northside Gardens, Sydney
Client: Jardine Bowen & Lipman
Architect: Kolos & Bryant

In 1964 we were fortunate to be associated with the winners of an architectural competition for a grandstand in Ipoh, Malaysia (Fig. 3). This brought us closer to the firm's Malaysian office and we have since enjoyed a close and interesting association with Arups in Singapore and Kuala Lumpur over a wide range of jobs. At roughly the same time we designed and supervised some wharves in the Solomon Islands which, although small, contained a great deal of engineering interest.

In 1966 we moved off the Opera House site bringing everyone together in a converted house in Walker Street, North Sydney, which we had bought for that purpose. The move from the Opera House made it easier to apply ourselves to the endless problems of that job with something nearer to objectivity. The old house, after some careful renovation and adaptation, provided us with a pleasant and intimate working environment till 1969 when it was sold and we moved into our existing offices in McLaren Street.

We had developed a reputation in the design of multi-storey buildings, which coincided with a property boom in the capital cities and gave us the opportunity to complete a number of unusual and interesting high-rise buildings including OCBC, Singapore (52 storeys); Capital Tower, Melbourne (42 storeys); Town Hall House, City of Sydney (24 storeys); Kingsgate Hotel, Sydney (34 storeys); North Point, North Sydney (38 storeys); Allendale Square, Perth (35 storeys) (Fig. 4). Numerous medium-height buildings were also designed, probably the most interesting being the Russell 14 Defence Building in Canberra and Northside Gardens in North Sydney (Fig. 5).

Fig. 6
East West Microwave Link
Client: Australian Post Office/
G.E.C. Australia Pty. Ltd.

Fig. 7
Boyle Street Town Houses,
Mosman, N.S.W.
Client: SOL Buildings Pty. Ltd.
Architect: Ancher Mortlock
Murray & Woolley

Our appointment for the structural engineering design of Hammersley House brought us to Perth in 1969. It was an auspicious start to a practice in Western Australia where a lot of work has since been done on structural and civil engineering projects. The office in Perth has been responsible for roads, drainage and sewage treatment in urban developments covering a total area about 1000 ha.

In 1966/67 we were associated with an extremely challenging job, a microwave link across the Nullarbor Desert linking Port Pirie in South Australia with Northam in Western Australia, a distance of 2250 km. The design and supervision of the foundations to meet the particular requirements of a remote and difficult area resulted in the development of a unique design solution (Fig. 6).

A client wanting our services for an office building in Melbourne was sufficiently encouraging to provide the stimulus for opening an office in 1972. Since that date the office has maintained a steady work load and completed the design of a number of interesting projects including Capital Tower, Bourke Street Office Development and is involved in the planning of Somerton Industrial Estate.

A continuing interest has been maintained in transportation engineering which has resulted in participation in many worthwhile projects including the North Sydney Development Control Plan, a comprehensive survey and the formulation of a development plan for North Sydney whose working population is expected to be 50,000 by 1980. Other transportation engineering projects include the suburb of Mosman, Sydney; the towns of Lae in Papua New Guinea and Bunbury in Western Australia.

An office in Port Moresby has been established recently to provide a service in highway engineering to meet the demands of the newly independent country.

Structural engineering has always been the mainstay and most important activity of the practice. During recent years there has been a swing of emphasis in the work from the private to the public sector with the largest project so far undertaken by the Australian office being the structural and civil engineering for the Westmead Hospital, an 1100 bed teaching hospital in the western suburbs of Sydney.

The CBA Bank Head Office in Sydney, a 39-storey building which includes some enterprising structural engineering developments, is presently under construction. The Australian Embassies in Bangkok, Singapore and Kuala Lumpur are all substantial buildings of architectural distinction and structural integrity. In all three we are responsible for structural engineering design and supervision.

A number of road and river bridges have also been designed.

Fig. 8
41 McLaren Street, North Sydney, Ove Arup & Partners offices
(Photo: Michael Andrews)

Michael Lewis established the first Ove Arup & Partners office in 1963 on the Sydney Opera House site. Since then the partnership has worked throughout Australia and South East Asia and the Pacific Region. Staff numbers in recent years have varied between 60 and 80 and projects whose engineering value exceeds A $40 million are currently being undertaken.

The firm has been associated with the following awards:

1970:
Canberra Medallion RAIA (to Ian McKay & Partners) for Woden Government Office Food Services Building

1970:
Canberra Medallion RAIA (to Collard Clarke & Jackson) for Russell Defence Offices

1972:
The 1972 Engineering Excellence Award of the Association of Consulting Engineers Australia for Glass Walls to the Sydney Opera House

Chronology

1963:
Sydney Opera House site office opened

1964:
Sydney Office opened. First commission received: Job A103 (Home Unit Project)

1965:
First work undertaken in Queensland: Job A105 (Paradise Towers)

First work undertaken outside Australia: Job A142 (Solomon Islands)

1966:
First commission in Canberra received: Job A164 (Russell Offices)

1968:
John Nutt made partner.
Canberra office opened by Dan Ryan

1969:
Perth office opened by Peter Thompson.
First work undertaken in Papua New Guinea: Job A461 (Lae Town Plan).
Traffic Group started by Jon Burgmann

1971:
Peter Thompson made partner.
First commission in Melbourne received: Job A535 (Bourke Street Offices)

1972:
Melbourne office opened by Peter Haworth

1973:
Michael Lewis returns to London.
Peter Thompson transfers to Sydney

1974:
First work undertaken in Bangkok: Job A819 (Embassy), Iran: Job A981 (Gombard Grandstand)

1975:
Port Moresby office opened by Colin Mathison.
Bob Kelman, Ian MacKenzie, Dan Ryan, Peter Haworth, Ken Gilbert made directors of Ove Arup Pty. Ltd.
First work undertaken in Indonesia: Job A986 (Siak River Bridge)

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