THE ARUP JOURNAL

DECEMBER 1981

THE ARUP JOURNAL

Vol. 16 No. 4 December 1981
Published by
Ove Arup Partnership
13 Fitzroy Street, London W1P 6BQ

Editor: Peter Hoggett
Art Editor: Desmond Wyeth FSIAD
Assistant Editor: David Brown
Australian editorial assistance: Diane Kell

Contents

Front and back covers: Blue Circle Southern Cement Works (Photos: Max Dupain)

The Australian practice: Foreword

John Nutt

This *Arup Journal* again brings together some of the work of the Australian practice. In the five years since we last compiled an issue the direction of the Australian practice has measurably changed. We continue to work with many of the best Australian architects but we have also a developed skill for projects in which engineering disciplines predominate – roads, bridges, transportation, telecommunication, ports, natural resources, utilities. We have as a result a rich kaleidoscope of work to which our people bring their particular talents arising from their background.

It might be asked why we wish to involve ourselves in such a range. It could be argued that a widening of projects involves a dilution of our technical skills and level of service. Not so. Superficially, the range is great. In application, however, the engineering knowledge required overlaps markedly with our traditional experience. But there is one significant difference.

Our work with architects brings us into contact with some of the most talented people in our community. They are in many ways the conscience of our built environment. Theirs is a difficult role. They need to create both economic function and environmental delight. The gifted ones do this with skill and facility. An engineer working with an architect must be both overtly receptive and technically proficient. While an architect should have inherently all the skills of building, or as Vitruvius says in the opening lines of his thesis – 'be equipped with knowledge of many branches of study and various kinds of learnings for it is by his knowledge that all work done by the other arts is put to the test'; this can rarely be so. Our supportive role in technical issues requires us to tread a wider stage. Flood risk analysis for a building in Canberra and mining techniques for a cross city underground arcade illustrate in these articles some of the ranges of engineering used to assist in the design of buildings on which we write. As architects benefit from our skills, so too do we gain from exposure to their sensitivity. As a result we bring to the engineering work in which we are masters an approach which reflects this community conscience.

Leading then from the notion that we are equipped and that we can contribute in a unique way, we have quite deliberately in recent years applied ourselves to work in traditional civil engineering spheres. We hope that the results you read of here demonstrate our proficiency and our approach.

The Australian scene is unique. From the urbanity of our cities to the desolation of our 'outback' the contrast is vivid and marked. Our projects extend across the country. There are some who will say a sensitive environmental approach is unnecessary in the remote areas of our country where many of the natural resource projects are sited. Few people will be there to be affected and in all probability they will have a connection with the project. It is in truth a very valid argument. But there are other arguments that also come into play. Those familiar with the eastern seaboard of the United States will know of the enormous conurbation in areas which earlier this century were wilderness. Or in the field of industrial relations, will not a worker in some of the thoughtless environmental developments consider himself a second class citizen in comparison with his head office counterparts? There is much to be gained in adopting a sensitive approach to even the unlikeliest of projects. And as you see, there are those who espouse this philosophy successfully, applying them to their business developments.

There is one other aspect which makes work in Australia interesting and challenging. It is an intensely competitive society. Australian, British, European, American, Canadian, African consultants and contractors all successfully operate in the construction field.

In spite of its small economic size, it provides an effective basis for a high technology firm. For this reason, when we have come to export our services we have found that we are highly effective and this is now such an increasing part of our work that a quarter of our fees come from overseas—in the Pacific, South East Asia and the United States.

Melbourne City Square

Peter Haworth

The design of Melbourne City Square is the result of an architectural competition held in 1976 and won by Denton Corker Marshall Pty. Ltd. Ove Arup & Partners were subsequently engaged as consulting civil and structural engineers. In the design development report, the architect analyzed the visual experience people associate with Melbourne and concluded that its essence was an open space system created as a by-product of individual buildings, each created with little consideration for other than the immediate neighbours, and frequently vying for attention. At its worst it is fragmented and unsatisfying, at its best it produces a sequence of attractive spaces, walks and varied experiences.

Melbourne's road system provides the city with a series of vistas, some formal, some otherwise, and within this context the Civic Square becomes a major formal space on the major axis of the city.

The square links the Town Hall and St. Paul's Cathedral at the heart of the city and is the area most readily identified as the centre of Melbourne. It fulfils two roles: Firstly, the formal civil aspect with the link between City Government and Church, well suited to ceremony because it opens off Melbourne's main street, Swanston Street. The other aspect is that of an informal square, attracting all types of people to participate individually, collectively or remain as bystanders.

The site to the east of the square is occupied by the Regent Theatre which was under threat of demolition to make way for commercial development. Public interest was aroused and led eventually to the retention of the theatre. It was then decided to incorporate the Plaza Theatre, situated beneath the Regent, into the scheme for the square, which was then able to function all year round with areas protected from wind and rain – not uncommon climatic conditions in Melbourne (see Fig. 3).

Levels

The commercial portion of the scheme is generally built on two levels. The main public meeting area of the square coincides with the lower shopping area. The upper level contains outdoor sales and an indoor tavern within the Plaza. The latter overlooks an air-conditioned garden lounge at the lower level. A visual link between the outdoor public meeting and commercial areas is provided by a water spillway connected through to fountains inside the garden lounge and to a large water feature in Fountain Place.

The architect was concerned about the shabby and overbearing effect of the 30 m high west wall of the Regent Theatre. A screen wall was built 3 m away from the Regent wall and was clad with sandstone similar to that from which the adjacent town hall and Anglican cathedral were constructed. A 15 x 6 m video matrix screen is incorporated at high level in the wall. This can provide transmission of programmes from any of Melbourne's five television channels, of events taking place in the square or from pre-recorded video tapes.

The scale of the new wall is visually reduced by the provision of a barrel-vaulted steel canopy which traverses the eastern side of the square at a height of 12 m above upper square level. Glazing covers the canopy over much of its length to afford partial weather protection and to suggest that the space is an extension of Melbourne's renowned shopping arcade network. The canopy is extended over Flinders Lane to the existing arch within the Chapter House of the cathedral.

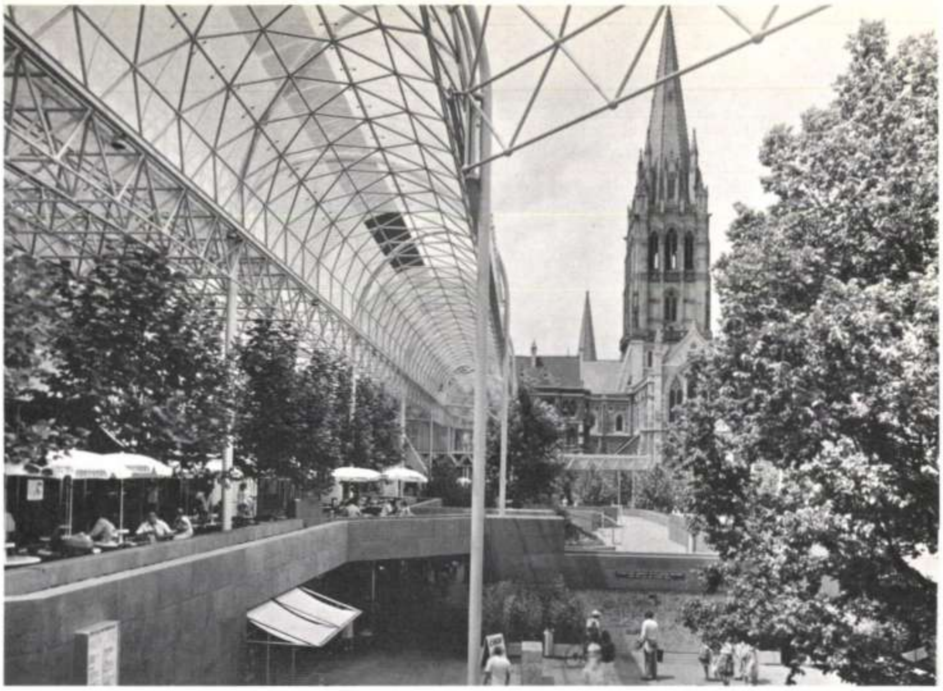


Fig. 1
Under the canopy looking towards the cathedral (Photo: John Gollings)

Fig. 2
Looking down into the square (Photo: John Gollings)

A flat glazed space frame extends from beneath the barrel vault at the south-eastern corner of the square to cover the amphitheatre where public entertainment is provided regularly. A two-level, air-conditioned restaurant is set between this space frame and the curved canopy.

A fundamental principle of the architectural design was the provision of a visual link between the town hall and the cathedral through the square. A formal axis is provided visually, linking the north transept of the cathedral to a major balcony window on the southern wall of the town hall. A reflecting pool is sited on the axis at the southern end.

The formal link was further improved when the Council agreed to the lowering of the levels of Flinders Lane and the Dean welcomed part-demolition of the existing north boundary wall to the cathedral.

The architect further identified the square with traditional Melbourne by using bluestone as paving and cladding throughout the scheme. This material is basalt, quarried and cut in the western areas of Victoria. The early buildings in the State were predominantly constructed from bluestone and recent significant developments in Melbourne have maintained this tradition.

General engineering works

The site is underlain by weathered Silurian mudstone. This material is typical of conditions on the eastern side of the city centre and is capable of sustaining bearing pressures of 600 to 800 kPa. Water levels were found to be approximately 1 m below the lower Plaza level. This necessitated tanking of the deep plant rooms beneath the lower square level. The material excavated during the construction was compacted to fill the various cellars of the buildings that had formerly occupied the site.

Reinforced concrete underpinning of the foundations to the southern portion of the Regent Theatre wall was required where this was situated adjacent to a deep facilities basement. Considerable difficulty was encountered in locating plant rooms and service ducts within the Plaza Theatre. Their levels were below those of the footings supporting the existing Plaza and Regent Theatre structure. The problem was exacerbated by

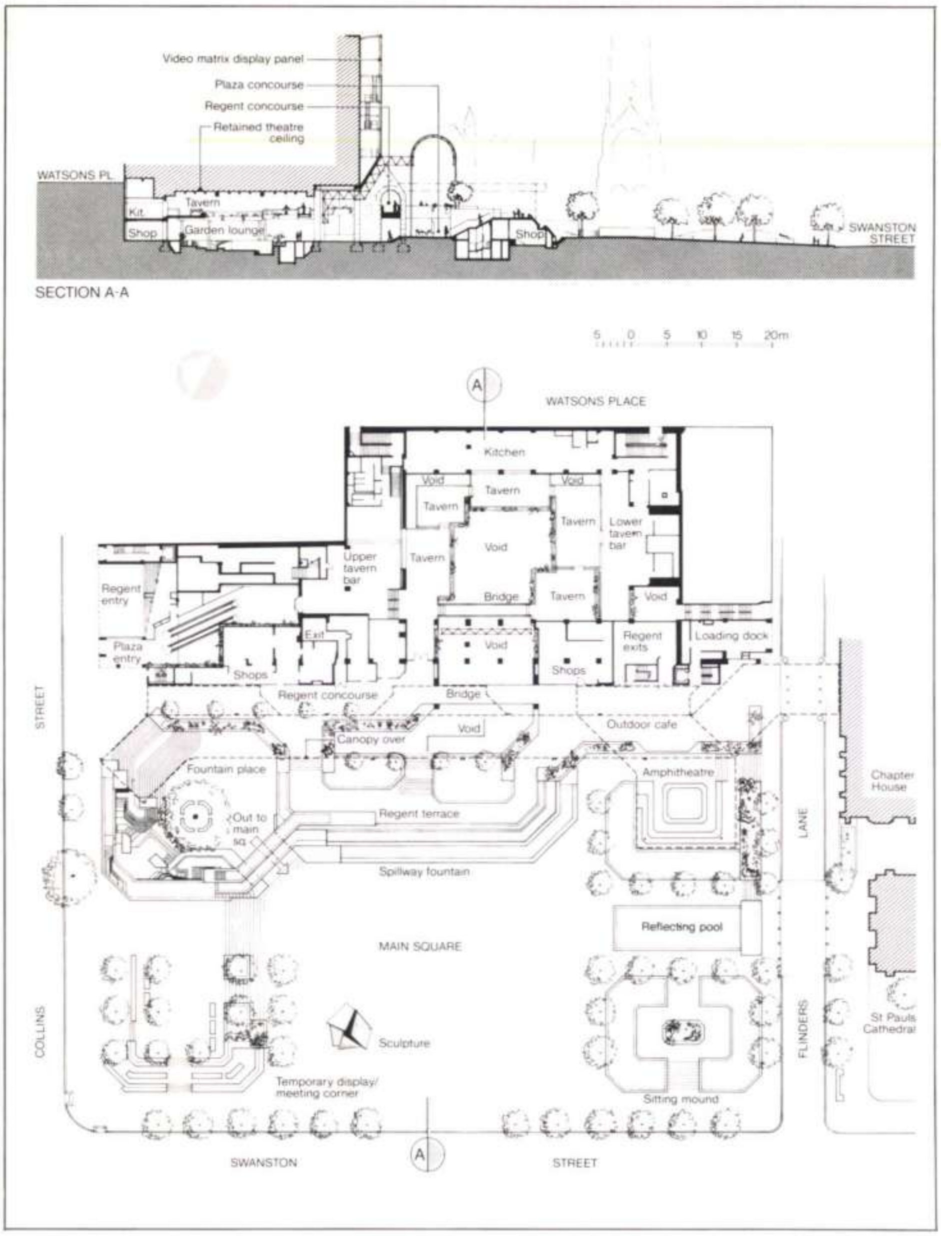


Fig. 3
Plan and section showing the connection between the areas beneath the Regent Theatre to the square

Fig. 4
Looking down on the amphitheatre roof and canopy
(Photo: John Gollings)

incorrect record drawings of the existing structure and it became necessary to replan many of the duct runs as construction proceeded.

Basement walls are waterproofed using an externally-applied bituminous felt coating. Walls, such as the water wall and aqueduct, are waterproofed on the water face using a similar bituminous compound and with an epoxy screed beneath the bluestone cladding.

Substantial steel underpinning of existing brickwork walls was needed in the Plaza in order to convert cellular structures into open areas within which shops could be incorporated.

The tavern level within the Plaza Theatre is constructed as a series of rectangular slabs at different levels – each supported by four corner columns. The latter are pinned at their junction with the slab edge beams to prevent moment transfer to columns, whose diameter had to be restricted to 300 mm for architectural reasons. Shallow perimeter beams 380 mm deep x 800 mm wide span up to 10.7 m between columns. These were cambered to counteract expected short and long-term deflections.

Stability of this series of table-like structures was achieved by interconnecting the individual slabs by reinforced concrete stairs and fixing a number of the slabs to the existing Regent Theatre columns.

The public assembly and seating spaces within the outdoor area were formed upon a 150 mm thick reinforced concrete slab supported directly upon undisturbed ground or compacted fill. 50 mm thick bluestone paving covers the concrete slab. Suspended slabs are designed as flat slabs supported by circular columns and various walls. Design loadings vary considerably depending on location. Live loads were taken generally as 5 kPa but many trees and planting areas were provided whose individual loads were assessed.

Stormwater drainage was designed to accept water from the Regent Theatre roof and the increased run-off from the hard surfaces of the new development. The system was connected into the main brick culvert running along Swanston Street.

A 200 mm thick reinforced concrete slab carries Flinders Lane as it passes through the square. Bluestone is used as a wearing surface and the panels are connected to the slab with brass dowels.

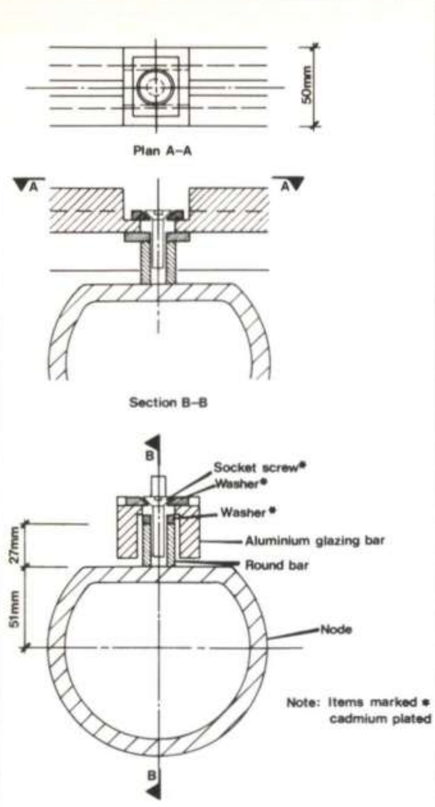
Structural steelwork

The screen wall to the Regent Theatre was constructed to the level of the video studio in reinforced concrete. A structural steelwork frame was formed above this level and is restrained by ties fixed to the original wall. A series of catwalks, service pipe supports, etc., run through this frame at different levels. The sandstone panel facing is attached to the steelwork frame using *Unistrut* fixings. Exposed steelwork is coated with mineral fibre to achieve the necessary low fire ratiug.

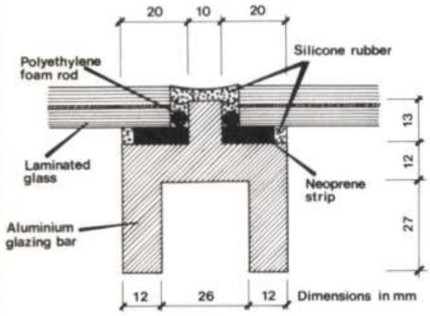
The geometry and layout of the barrel vault and space frame areas were fundamental to the competition scheme and the architect expressed a preference for tubular steel construction using a standard tube size of 60 mm outside diameter (o/d) throughout.

In general this was achieved, although some tube diameters had to be increased near the supports to the space frame forming the amphitheatre canopy and restaurant floor. The structure of both the barrel vault and its supporting 168 mm o/d arches is arranged on a grid set at 45° to their common longitudinal direction.

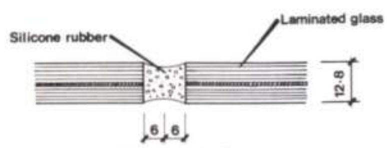
Whilst proprietary space frames were considered, a custom built frame was found to be the most economical in this instance. The sub-contractor chose to adopt a node in the



a) Typical connection between node and glazing bar



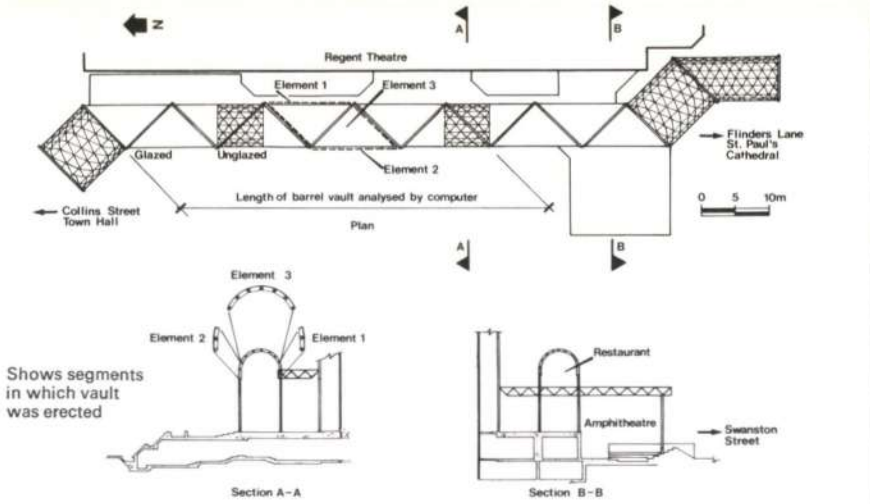
(i) Glazing bar joint



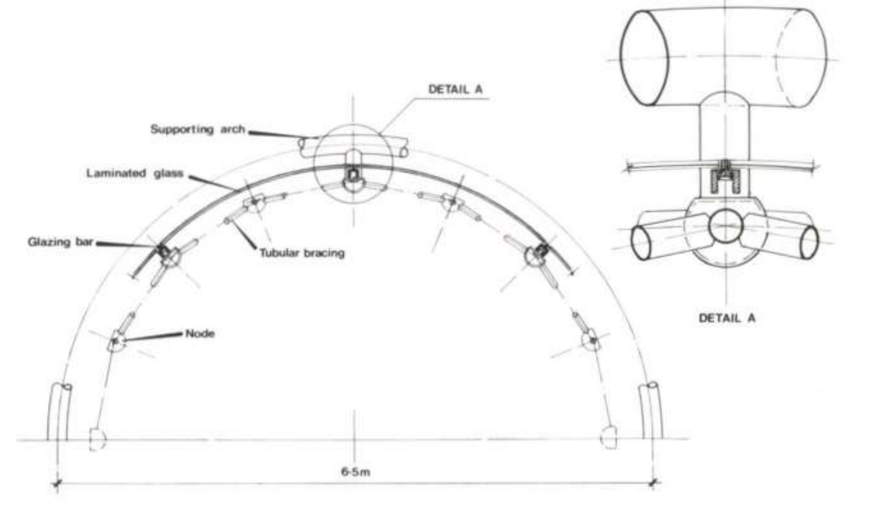
(ii) Butt joint

b) Typical glazing details

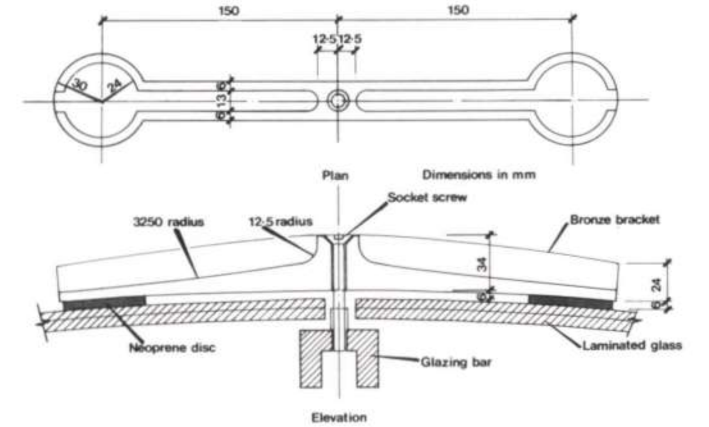
Fig. 5
Details of space frame



c) Layout canopy structure



d) Typical section through barrell vault



e) Details of external fixing brackets

form of a 135 mm o/d hollow sphere of 10 mm wall thickness. These nodes were cast in two halves and then welded together. The ends of all tubular members were square cut and welded to the nodes. Laboratory tests were carried out to confirm the tensile capacity of the nodes.

Individual computer analyses were carried out on a 52 m length of the barrel vault and its supporting arches and also on the amphitheatre spaceframe. These analyses were used to establish the worst combinations of shear force, bending and torsional moments and from these the capacity of the individual members was checked by hand. In particular the buckling capacity of each of the long supporting arch columns was examined in detail. The tendency for the vault to splay was resisted by the inclusion of a horizontally oriented channel on the east side (used as a rainwater gutter). On the east side one complete spaceframe bay was included to carry the outer wheel of the window cleaning gantry. The vault was regularly tied back to the Regent Theatre wall, thus preventing undue dynamic movement and horizontal deflection of the structure. Individual internal and external window cleaning gantries were designed to traverse the length of the vault. Both of these were constructed as tubular truss structures such that their appearance is compatible with the canopy framework. The barrel vault and spaceframe were prefabricated into large segments prior to transport to site. Each length of vault was fabricated as two vertical skirt sections and an upper arched section. The length was normally 13 m and the height of the arch 2.6 m above the springing points. The large supporting frames were also fabricated in three similar units. Each lattice unit was connected to its neighbour by welding adjacent half nodes following attainment of final positional adjustment.

All tubular steelwork was blast cleaned and primed with zinc rich epoxy paint. Final coats of polyurethane undercoat and gloss paint were added in situ before commencement of the canopy glazing.

Glazing

The architect required the glazing material to be transparent and to have a long-term surface sparkle. Various plastic materials were considered – as were thermally toughened and

Fig. 6
Water garden

Fig. 7
Internal seating area shows the decorative ceiling of the former Plaza Theatre

(Photos : John Gollings)

chemically toughened glass. Ultimately it was decided that laminated glass would be the most appropriate material and this was adopted. Clear glass was used throughout except in the flat glazed areas between the barrel vault and the Regent Theatre wall where a silver reflective coating was incorporated.

The external appearance of the vault canopy was envisaged by the architect to be in the form of a continuous glazed surface suspended beneath arch supports. The glass panels are held in position using a minimum of support. Opposite edges are seated on extruded aluminium glazing bars and the upper surfaces are clamped to the glazing bars by discrete patch fixings made from chrome plated bronze. Similar details were used on the amphitheatre canopy. Translucent silicone rubber was used to waterproof the canopies both at the glass/glass and glass/glazing bar joints (see Fig. 5).

John Hooper in the Research and Development Group carried out the design and testing of the laminated glass. The Uniform Building Regulations of the State of Victoria required that the flat canopy should be capable of supporting a mass of 90 kg placed centrally between supports without the occurrence of fracture. Also, purlin spacings should not exceed 1.25 m and mesh should be provided beneath the glass. The amphitheatre glass spans 1.4 m between glazing bars and that in the barrel vault is 1.6 x 2.4 m (curved length) with a radius of 3.1 m.

These sizes meant that it was not possible to comply directly with the Building Regulations whilst achieving the architect's aesthetic requirements. It was therefore decided to prove that the glass properties satisfied the intent of the regulations.

The flat glass was designed to withstand a live loading of 0.25 kPa or a localized 90 kg mass. The curved glass was designed to accommodate a live loading of 0.25 kPa. A basic design wind pressure of 0.4 kPa was used with a total drag coefficient of 1.5 made up of 1.0 on the windward face and 0.5 on the leeward face. Local pressure increases were applied at the bottom and ends of the panels.

Preliminary hand calculations and subsequent finite element analyses showed that two 6 mm sheets separated by a 0.76 mm thick polyvinyl butyral laminate would be adequate.

Static loading tests showed that glass fracture did not occur until a load of 128 kg had been applied to one edge midway between supports. Moreover there was sufficient residual strength in the fractured panel to sustain the 128 kg mass for several minutes, after which the panel was unloaded. Finally impact tests were performed which showed that the panels accorded with the requirements of safety glass issued by the American National Standards Institute. In these tests the performance of the glass panels was observed when impacted by a 45 kg shot bag swung in a pendulum arc to strike the centre of the test panel. The successful completion of the tests led to the Building Regulations Committee issuing the necessary modification allowing construction to proceed in accordance with the architects' requirements.

Conclusion

Construction of the City Square scheme was completed at an approximate cost of $14m. and was formally opened by Her Majesty the Queen on 28 May, 1980. Inevitably it has attracted both praise and criticism – particularly in regard to its yellow steel sculpture – but the public uses its facilities in great numbers and that surely is the ultimate object of an urban space.

Credits

Client:
Melbourne City Council
Architect:
Denton Corker Marshall Pty. Ltd.
Main contractor:
Dillingham Australia Ltd.

Blue Circle Southern Cement Works

John Nutt
Bill Thomas

The relationship now existing between Blue Circle Southern Cement and Ove Arup & Partners started with a telephone enquiry to the firm as to whether we 'know anything about concrete'?!

This was followed by a meeting at which we were given the commission for the structural design of the Berrima pre-heater tower and raw meal silo. There was a very tight programme for the documentation of these sections of the plant, which we were able to meet. This undertaking was in fact a principal factor in our favour in obtaining the commission. The civil commission developed to include the coal blending plant, clinker silo, conveyor support structures, a steel chimney, and being principal agent involved architectural, costing and landscaping advice.

It was then extended to providing assistance on contractual matters for a number of the mechanical plant contracts. We had in our team Peter Hall, with whom we had previously worked when he was design architect on the Opera House, and Rawlinson Edmonds, quantity surveyors for cost control.

The Blue Circle Southern Cement works at Berrima, NSW, is the major supplier of cement in that State. In May 1979 the capacity of the plant was increased by 750,000 tonnes per year following the completion of new facilities at a cost of $58m.

The new installation manufactures clinker by the dry process. Limestone and shale are blended together and ground to form raw meal which is then heated by the exhaust gases from a revolving kiln in a series of cyclones before entering the kiln. Pulverized coal is burned and fed into the kiln to meet the raw meal which is converted by the heat to clinker. The clinker is then railed to the BCSC works at Maldon for grinding to produce cement.

BCSC have been producing cement at Berrima for 50 years. The works is situated in the very beautiful countryside of the Southern Highlands of NSW where it is the major industry. It was important that the new extension should be well designed. Two of the structures, the preheater tower and the raw meal blending silo, are dominant features in the landscape.

Fig. 1
BCSC works at Berrima, NSW (Photo: Max Dupain)

BCSC believe that concrete should be used for the major structural systems. Not only should its own product be used, indeed it should be employed so as to set an example for concrete structures in industrial complexes.

There can be no question that the efficiency and performance of the manufacturing process are the paramount factors in the design of industrial plants. The size of the operations can be so immense, and the consequences of failure so great, that the equipment can only be designed by experienced manufacturers, highly specialized in one or more aspects of the process. Such manufacturers design similar equipment for a range of industries, to be selected and arranged in juxtaposition to give the best performance. The scale of these pieces can be such as to be major structures in their own right. It is not possible as a result to have either visual unity of individual components or consistency of the plant as a whole.

The civil work in an industrial complex must be subservient to the mechanical engineering and equipment. Support structures must be completely functional. The efficiency is not to be compromised. Nevertheless, the design of the structural components within the strict mechanical discipline can influence the visual character of the plant. Indeed, it is only through the design of these secondary structures – buildings, towers, supports – that the complex can be given any unity.

The major concrete structures at Berrima comprise the preheater tower, the raw meal blending silo, conveyor supports and the coal-blending building (see Fig. 2).

Preheater tower

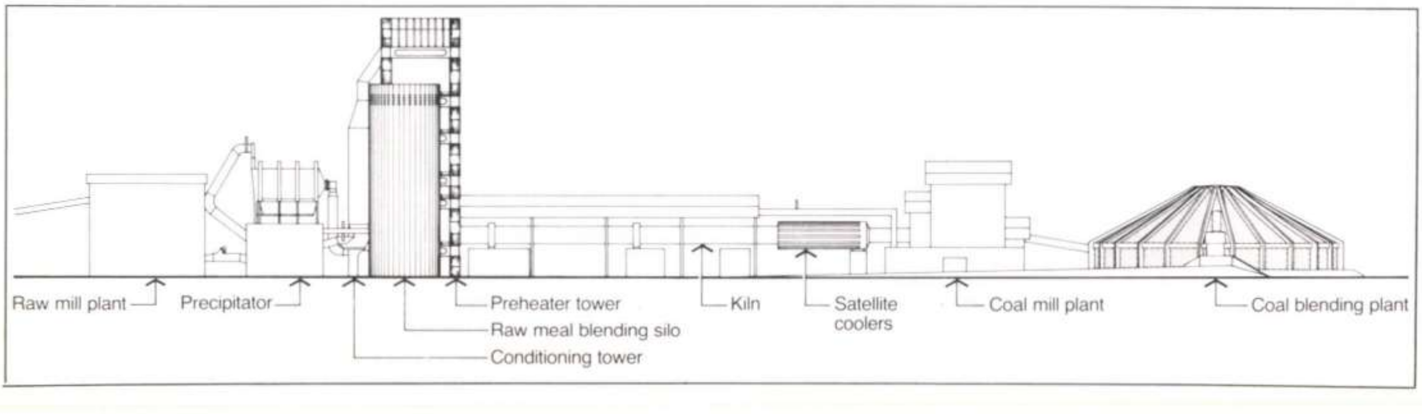
The preheater tower is some 70 m high and contains cyclones in which the raw meal is heated by exhaust gases from the kiln, reaching 900° C prior to entering the kiln, from which the clinker emerges. The planning of the concrete tower allows ready access to all parts of the multi-level floor system. Clear spans give complete freedom for future alterations and maintenance.

Four large fluted corner columns provide the vertical support system. Deep shaped edge beams span the 22 m between columns (see Figs. 3 and 4).

The structure has been conceived on a sufficiently large scale so as not to be dominated by the ducting and equipment which it supports, while precise detailing shows the concern and consideration that has been taken with the planning.

The subsurface condition is fill below which is firm sandy clay. This in turn is underlain by 1–5 m of weak to medium strength shale

Fig. 2
Diagrammatic section through works



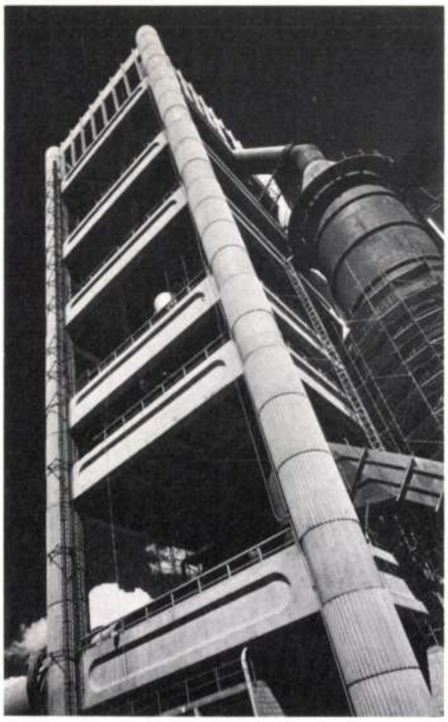
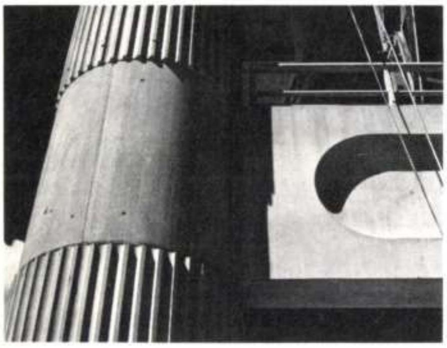


Fig. 3
Preheater tower

Fig. 4
Detail of preheater tower
(Photos: Max Dupain)



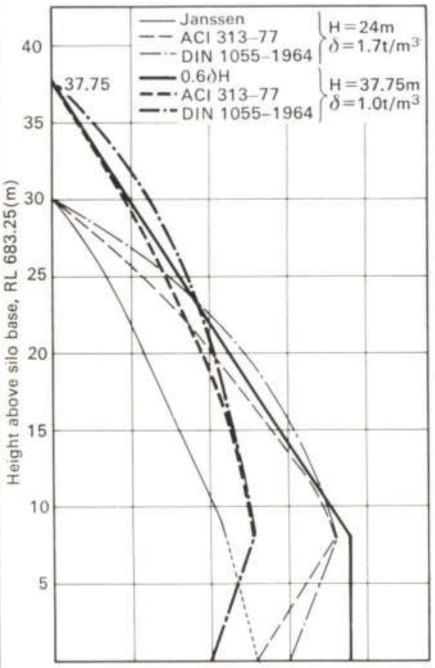


Fig. 5
Comparison
of wall pressures

with interbedded sandstone and siltstone followed by fresh shale.

The foundations of the preheater tower comprise three 1050 mm diameter in situ concrete piers supporting each of the four corner columns. All piers are socketed into fresh shale and designed for 5000 kPa allowable end bearing and a shaft adhesion of 800 kPa.

Each suspended floor, in addition to its self weight, carries approximately 300 tonnes of fixed plant together with an allowance of 11 kPa superimposed live load. Earthquake loading is in accordance with that for Zone 1 of the Australian Draft Earthquake Code. When compared with wind loading, earthquake loads controlled the design of the structure resulting in 2.6% of floor loading applied horizontally at each level.

Raw meal blending silo

The finely ground raw meal is transported into the top compartment of the silo where, through eight discharging ports, it is fed into the main chamber. Air under pressure is introduced through a series of airslides located over the full cross-section of the silo base, activating the material and causing it to expand and break down the close contact between the particles. It is blended to an exact chemical composition. These airslides are used for extraction.

A conical expansion chamber within the silo allows the material to be ventilated before discharging through a side exit into the preheater. The contents at this stage are at a high temperature. The design temperature adopted was 120° C, that of the meal leaving the raw mill.

The 8,000 tonne capacity silo is 18.0 m in diameter and 53 m high. The outside of the concrete walls, which vary in thickness between 450 mm at the base and 300 mm at the top, has been fluted to give a visually acceptable form. The walls were cast using a self-climbing static form which has given an excellent finish to the concrete (see Fig. 11).

The structure is supported on 15 1050 mm diameter in situ concrete piers evenly spaced around the perimeter wall of the silo together with seven piers supporting a central column which in turn supports the silo base slab.

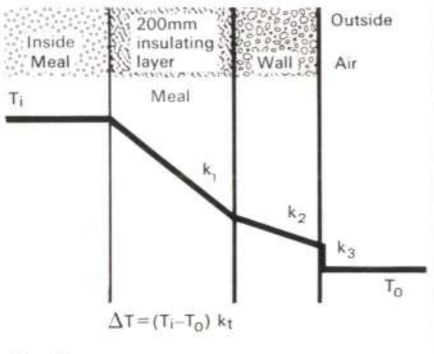


Fig. 6
Idealized heat transfer diagram

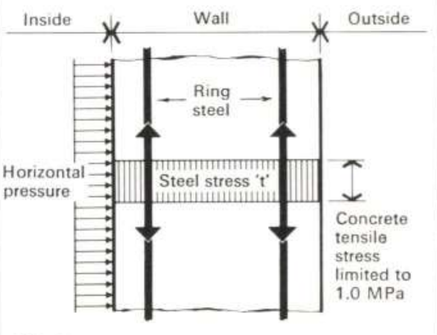


Fig. 7
Idealized stress diagram for computer analysis – plan section of wall

The design parameters – material density, wall pressures and temperature effects on the silo wall are issues of great debate in the industry. The original classic theory of Janssen has been shown to be deficient in practice and many failures have been recorded. The material codes of the United States, Germany, France and the Soviet Union recognize this but all give somewhat different results.

There is no reliable method of measuring density of the powder during operational conditions. The industry uses a variety of methods based on experience. Most common is the compaction box to which is applied formula to account for the increase of density with head and method of loading. Promising methods are being developed in Australia, based upon the principle of gamma radiation, but are not yet operational.

A range of densities was given to us by the process engineers varying from 0.7 tonnes/m^3 in the fully aerated condition to 1.7 tonnes/m^3 in the fully compacted condition, which is the value given for cement in the German Code *DIN 1055*. Our client subsequently carried out laboratory compaction tests recording 0.98 tonnes/m^3 for packed, 1.18 tonnes/m^3 for fully compacted. The operational density was measured during initial filling of the silo and conforming to an as found state from an average height of 30 m, was 1.1 tonnes.

The capacity of the silo with raw meal in a fully compacted condition is much greater than its rated capacity of 8000 tonnes. The process engineers considered this an unrealistic design criterion. We selected with them two alternatives – 8000 tonnes in the silo, fully compacted by occupying partial volume, and completely filled in an aerated condition with a density of 1.0 tonnes/m^3. After the silo was in operation and we had more information on the operational density of the meal, we increased its rated capacity to 10,000 tonnes.

It is well accepted now that wall pressures are very much greater during emptying than when filling or under static conditions. The pressures upon emptying vary according to the type of flow pattern that occurs. Core flow occurs when the bulk solids discharge through a central vertical pipe which forms within material in the bin leaving standing a large perimeter annulus adjacent to the walls. This mode of flow occurs when the walls are rough and the angle of the hopper bottom is too flat.

Flow pattern

Mass flow is the ideal flow pattern. The material is in motion at substantially every point whenever material is drawn from the outlet. Flow occurs along the wall of the silo and the surface moves down relatively uniformly. There is little dead capacity within the silo. Mass flow results when the walls are sufficiently steep and smooth. The greatest wall pressures occur during these conditions.

Wall pressures were determined by the methods recommended by the US Code *ACI 313–77* 'Recommended practice for design and construction of concrete bins, silos and bunkers for storing granular material.' By this code, two design cases must be examined and the larger assessed pressure controls. One case applies an overpressure factor to the Janssen theory – in this case 1.75 – to account for high pressures which occur during emptying. The code notes that these loads are 'inadequate for the higher loads associated with mass flow'. The other case applies solely to homogenizing silos where air pressure is used to mix the powder. The mixture may behave as a fluid and then the probability of hydraulic

Fig. 8
Conveyor support shaft
(Photo : Max Dupain)

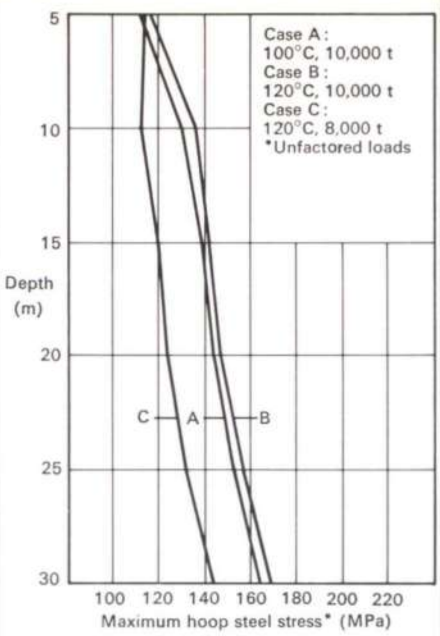


Fig. 9
Comparison of maximum hoop stresses

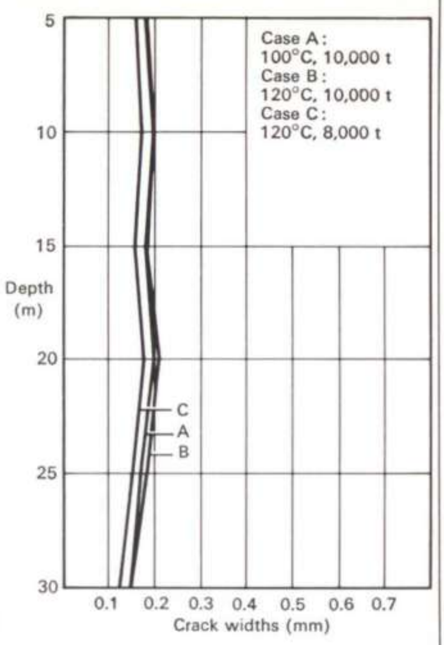


Fig. 10
Comparison of crack widths

Fig. 11
Raw meal blending silo
(Photo : Max Dupain)

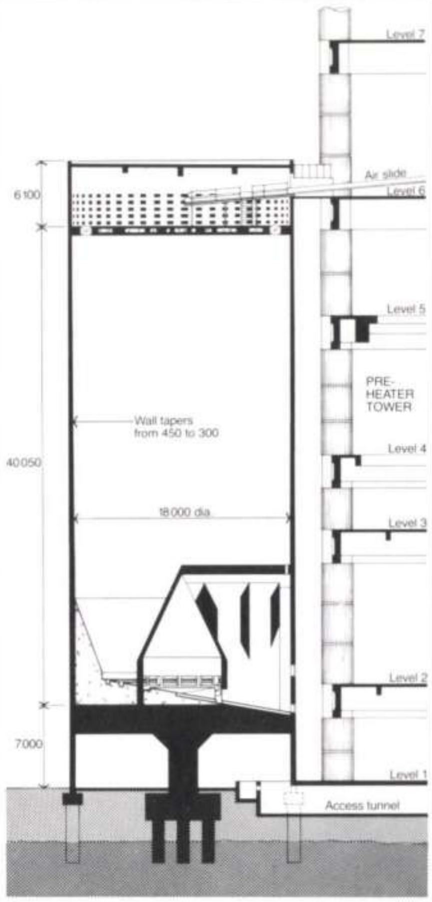


Fig. 12
Section through raw meal blending silo

pressures must be examined. The horizontal pressures are calculated from 0.6 times density times height. The 0.6 factor reflects the fact that suspended particles are not in contact and the average density is less than the material at rest.

An envelope of horizontal pressure was calculated for these two load cases combined with the density and capacity criteria referred to previously. These are shown in Fig. 5 on which is also shown pressures determined by Janssen for reference and by the German Code, *DIN 1055*.

A base silo wall thickness of 450 mm was calculated from an empirical formula in ACI, and it was decided to taper the walls to a minimum of 300 mm at the top. Hoop steel was designed to a stress of 175 MPa for cold worked bars.

The friction load on the silo wall was calculated from the Janssen theory. This load plus those from the roof and distribution slabs, the silo walls self-weight and 50% of the base slab (which was assumed to carry the full 8000 tonnes) was summed as the total vertical load on the wall base. Only nominal steel was required and a ratio of 0.002 was adopted.

Earthquake loading was calculated on the basis suggested in the draft Australian Standard. This amounted to the application of 2.6% of the vertical loads to be applied horizontally. Two cases were examined, dead load + live load + raw meal (8000 tonnes) and dead load only. Moments were taken about the pile cap and in both cases no tensile stresses were noted.

Wind loading amounting to 1.5 kPa over the silo face resulted in bending moments less than those for the earthquake loading.

The method adopted for determining the temperature steel for a maximum raw meal temperature of 120° C is that presented by S. S. Safarian and E. C. Harris in the *ACI Journal,* July 1970.

It assumes a temperature gradient across the thickness of silo wall based upon the heat transfer occurring through an insulating layer of material at the wall face, and the wall (see Fig. 6). Their recommendations of subtracting 44.5° C from the overall temperature difference is dubious and was not adopted. Initially the resulting ultimate bending moment was derived from an uncracked section whereas the added reinforcing was chosen for a cracked section.

For the vertical steel the governing case is the steady state temperature gradient just above the meal where the air (at meal temperature) is in direct contact with the wall. For the hoop steel the maximum temperature gradient was coincident with the maximum meal load effects. This is because the overall cooling rate of the mass of meal is likely to be so slow that by the time filling is complete the temperature has fallen little.

Transient temperature conditions soon after placing the hot meal next to the wall were not important because the equivalent straight line gradient for non-linear temperature distribution was always less than the steady state case. The transient temperatures do give higher compressions on the hot face, but these were not significant at operating temperatures.

Once the hoop and temperature steel had been selected, the steel stresses resulting from the various load conditions were checked via a computer program developed in London by John Blanchard. A similar approach to calculating the temperature difference, i.e. assuming heat transfer through a 200 mm thick insulating layer of material and also ignoring the adoption of a base temperature moment, should be calculated from the uncracked I, the cracked I, or from some intermediate value of I. The

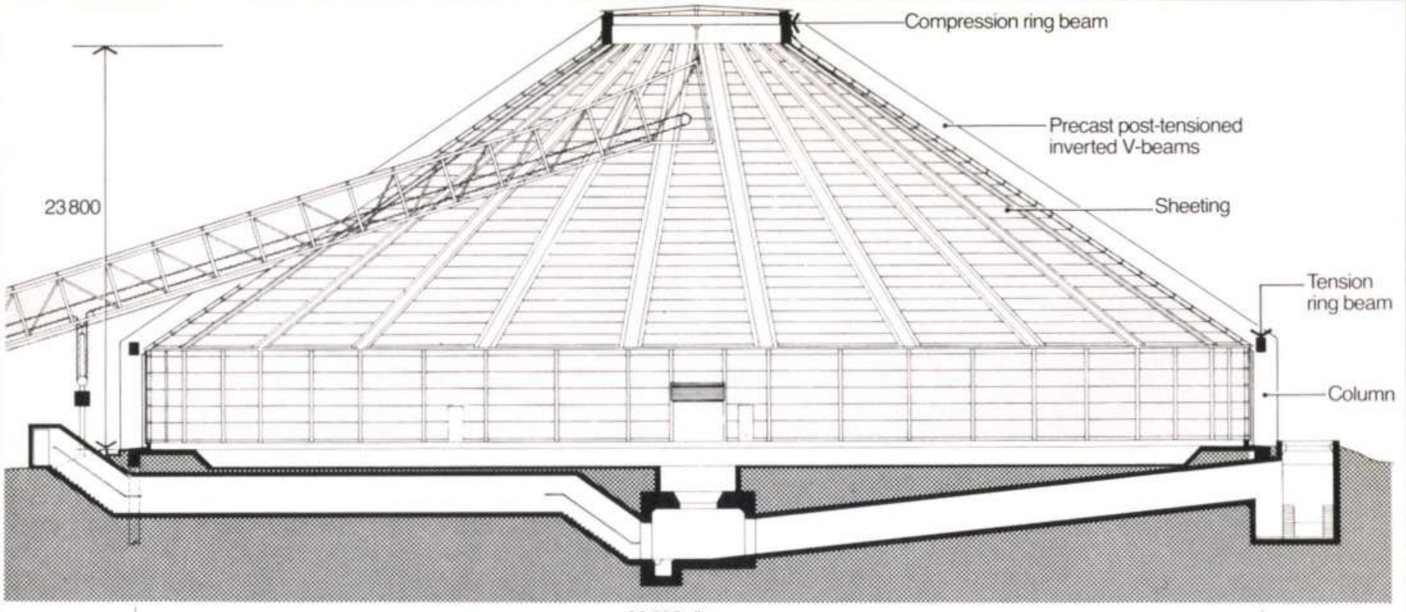


Fig. 13
Section through coal blending plant

(Safarian and Harris) method, assumes an uncracked value for I, whereas the usual chimney formulae implicitly assume a value of I for the section cracked to the neutral axis. The first method can give bending moments up to 15 times that of the second, but neither is correct since neither takes true account of the stiffening effect of the concrete between cracks.

The method adopted for this program is that suggested in *CP 110* for finding the deflections of a cracked beam. The program was developed assuming an average strain diagram which varies linearly through the thickness of the wall and is consistent with the temperature gradient. In addition the stress distribution across the section implied by this strain distribution must be in equilibrium with the ring tension. It was in the calculation of these implied stresses that the device of *CP 110* was introduced, i.e. the stress is taken as Ec x strain, provided that a tensile stress does not exceed some limiting value 't'. The program took a value of 't' between 0 and 1.5 MPa, the actual value chosen to maximize the steel stress (see Fig. 7).

These criteria determine the average strain distribution from which the average stress in each ring of steel could be found. However, at a crack, no tensile stress can exist in the concrete so that the tensile stresses have to be replaced by statically equivalent forces in the two rings of steel. The steel stresses due to these forces are then added to the average steel stress to obtain the maximum steel stress.

Crack widths were calculated from two formulae in *CP 110* and a weighted average chosen as representative. The tensile strain at the outer face was required and this was calculated in the program just discussed.

Results of steel stress for the vertical steel indicated that the nominal steel suggested by ACI was adequate. After further investigation of the actual operating temperatures, the design temperature for raw meal entering the silo was lowered to 100° C. The results for the hoop steel stresses and crack widths for both the original (8,000 tonne capacity silo) and the new (10,000 tonne capacity silo) design cases are shown in Figs. 9 and 10.

Re-analysis via the computer program, developed earlier, led to steel stresses and crack widths for the walls which were still within the allowable limits. It is worth noting that temperature has a small effect on crack widths because the cracks due to pressure hoop tension must close on the inner face before the temperature gradient can produce significant stresses.

As construction neared completion we were again able to help with the commissioning of the plant by monitoring temperatures and movements of the raw meal silo, thus allowing increased rate of filling and ultimately, by re-analyzing temperature-related stresses, were able to justify increased capacity of the silo. Cracks observed were consistent with those predicted. Horizontal cracks opened and closed as the level of the raw meal moved past.

Conveyor supports

Conveyors link every major component of the complex. The supports range in height from 3 m to 17 m. In all there are some 16 conveyor supports of which seven have belt takeups incorporated in them.

A family of structures has been designed: both the top and base are standardized and the intermediate fluted sections accept the change of height and function.

Coal blending building

The coal blending plant handles 7,000 tonnes of continuously blended coal for the firing of the kilns used in the production of cement clinker. A revolving reclaimer pivots about the centre and requires a building of some 66 m clear span.

A concrete building of this span is too heavy and too expensive to be justified for an industrial plant. Nevertheless, BCSC's wishes have been complied with by adopting exposed concrete ribs and columns which show the character of the building, infilling between the lightweight metal roofing.

24 roof beams, in the form of an inverted 'V', span from a compression ring at the crown to a tension ring at the eaves. The perimeter ring has been post-tensioned to take the thrust. Each roof beam weighs 39 tonnes and has been precast in three 10 m segments from the same steel mould.

To cast matching faces, the two end units were located at either end of the mould, lined up and the central segment cast. The three segments are joined to form a roof beam by post-tensioning across thin fluid epoxy joints in a variation of the technique originally pioneered on the Sydney Opera House. Thus, immediately prior to stressing, the interface was coated with epoxy resin – CIBA-GEIGY *Araldite* and *LC 125 Hardener*.

Each precast roof beam has four cables comprising 14 15.2 mm diameter super grade, stress-relieved strands stressed to give a total jacking force at transfer of 2976 kN. The VSL system was used for all stressing works. The precast tension ring has two cables comprising 20 12.5 mm diameter super grade, stress-relieved strands stressed to give a jacking force at transfer of 3120 kN.

Each of the 24 columns is supported on a single 700 mm diameter in situ pier, socketed into fresh shale. The tops of the piers are connected by a circular ground beam.

A central scaffold was erected up to the crown of the roof to support the construction of the in situ ring beam and to offer temporary support for the roof beams. A single P & H crane, model 625TC, was used to lift the units from a location outside the building. The erection of the concrete ribs was completed in one week. Cold formed metal purlins span between the ribs to support the roof and wall-sheeting.

The complex won for Blue Circle the 1979 Award of the Australian Institute of Materials Handling. Conveyor systems are used for all materials handling. The judges specifically commented on the efficiency of the complex 'which reveals a high degree of planning and purposeful application'. The major contractor for the materials handling systems was Noyes Bros. Pty. Ltd., whose contracts covered raw materials, preparation, blending, storage and conveying, clinker conveying, storage and loadout and the coal receival, blending and blending systems.

The Association of Consulting Engineers of Australia awarded Ove Arup & Partners its 1979 Merit Award for Structural Engineering and the structures also received the 1979 Concrete Institute of Australia Award of Excellence.

Credits

Client:
Blue Circle Southern Cement Ltd.

Principal agent:
Ove Arup & Partners

Consulting architect:
Peter Hall

Quantity surveyor:
Rawlinson Edmonds

Contractors:
Preheater tower and raw meal blending silo;
Citra Construction Ltd.

Coal blending building and conveyor supports;
The Hornibrook Group, Southern Division

Vanuatu broadcasting services project

Colin Mathison
Peter Thompson

'HIS EXELENSY ATI GEORGE SOKOMANU, PRESIDEN BLONG RIPABLIK BLONG VANUATU HEM I OPENEM BRODKAS HAOS IA LONG TASDE NAMA 30 APREL 1981. HAOS IA, GAVMAN BLONG OSTRELIA I GIVHAN LONG HEM ANDA LONG "AUSTRALIAN DEVELOPMENT ASSISTANCE PROGRAMME".' So reads the plaque on the new studio which was officially opened on 30 April 1981 with the inaugural broadcast given by the Prime Minister, the Honourable Father Walter Lini.

Father Lini heads one of the world's newest and smallest states which, in its first year of independence, successfully weathered a rebellion with the help of neighbouring Papua New Guinea's troops.

The independent Republic of Vanuatu, formerly the Condominium of the New Hebrides, lies in the South Pacific some 600 miles west of Fiji and 1200 miles off the Australian mainland. The archipelago comprises 110 beautiful tropical islands – palm trees, coral reefs, white beaches, blue seas and jungle covered mountains. Only 70 of the islands are inhabited and half the population of 120,000 lives on the four main islands of Efate, Espiritu Santo, Malekula and Tanna. The capital, Port Vila, is on Efate and Luganville on Espiritu Santo is the other major town (see Fig. 1).

The islands have a colourful history. Discovered by Pedro Fernandez de Quiros, one of the last of the great Spanish explorers, in 1606, the country came under joint administration of Britain and France in 1907. But it was remote from western civilization and 'blackbirding' – the taking of indentured labourers, sometimes by force, for work on the Australian sugar farms – was rife at the turn of the century. As recently as the 1920s American filmmakers were making documentaries with such trite titles as 'Among the cannibals of the South Seas'!

Independence

Now the nation is independent but the English/French legacy is a trilingual society – English, French and Bislama, with French restaurants in the main towns. The Condominium was unique in the world's governments – joint decisions by the colonial powers. But the bureaucracy was a civil servant's dream! Both Britain and France had duplicate administrations which had created separate education, police and health systems. Some departments were shared – Communications and Public Works – but these rarely had philosophical consistency because heads of departments were alternately French or English. In this administrative nightmare a tax haven flourished and alongside the more agricultural pursuits of growing copra, timber and cocoa, the accountancy and legal professions prospered.

The broadcasting system was the first of three similar projects Ove Arup & Partners are undertaking for Australian Aid in the newly emerging nations of the South Pacific. Others are in the Solomon Islands and Nuie. The value of the aid for this project was A$1,750,000 (£0.75 m.).

In early 1977 we were approached by two established radio engineering consultants to head a consortium to undertake a feasibility study for broadcasting services, telecommunications and civil aviation radio

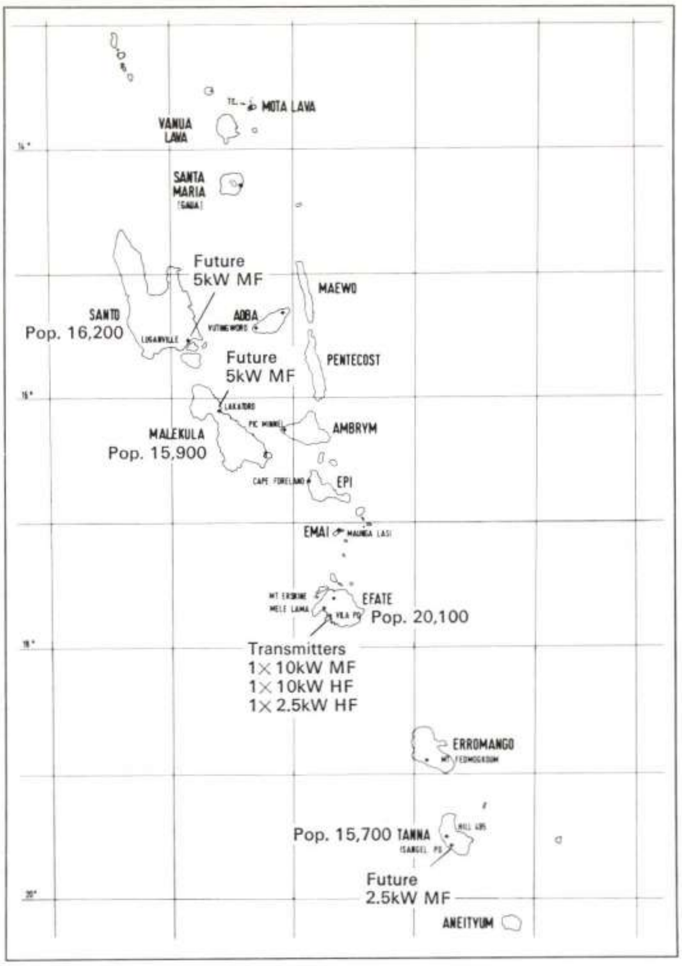


Fig. 1
Vanuatu Broadcasting Services transmitting plan

Fig. 2
Radio Vanuatu Studio (Photo: K. Woolley)

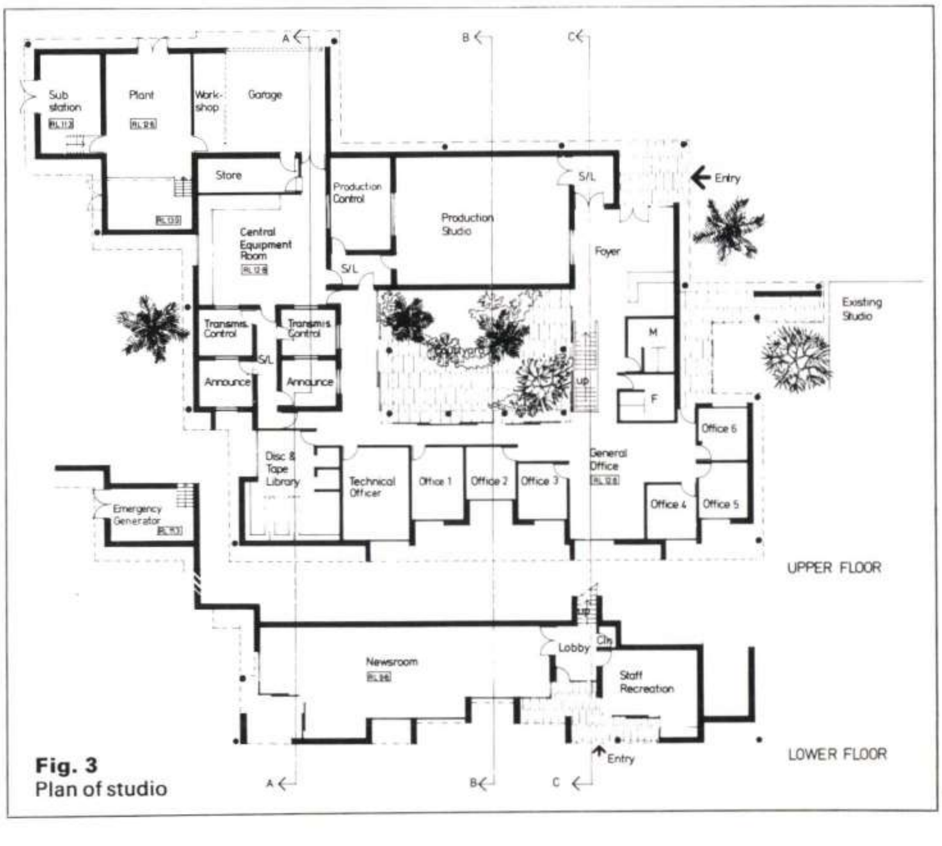


Fig. 3
Plan of studio

Fig. 4
Studio building showing the emphasis on sun shading to windows and rain water dispersal (Photo: H. Calverley)

Fig. 5
Studio equipment
(Photo: R. Thyer)

navigation aids in the then New Hebrides. This feasibility study was to be commissioned by the Australian Development Assistance Bureau (ADAB) which is a department of the Ministry of Foreign Affairs, charged with administering Australia's foreign aid programme.

We responded in detail to the terms of reference and to our great surprise were successful. We believe we were helped to some degree because other organizations who submitted were in some way connected with the radio manufacturing industry whereas we were a purely professional and independent group.

At an early stage the telecommunications study was deferred but more of that perhaps in the future.

The basic requirements for broadcasting as defined in the terms of reference were to provide a useable signal at either medium frequency (MF) or high frequency (HF) to the country and to establish studio facilities in the capital, Port Vila, and satellite studios in other centres of population where necessary, all within reasonable constraints of geology, geography and budget.

Early limitations

On our first visit we found the existing broadcasting system was almost dying on its feet from lack of proper equipment and facilities. One very small homemade studio existed in Port Vila to produce and transmit programmes in three languages with support staff at a remote location. Actual broadcasting time was limited to four hours per day.

Transmission facilities were equally as bad. These were also located in Port Vila and consisted of a 30 year old 2.5 kW transmitter operating at 1 kW power output on 1420 kHz (MF) and a rather more modern 2 kW transmitter operating on 3945 kHz (HF) at night and 7260 kHz by day.

The aerial systems were in bad order with the ground mat under the MF radiator almost non-existent and the HF system badly out of alignment.

The result of this was that the MF transmission was marginal in parts of Port Vila itself and the HF service was unreliable due to propagation difficulties. One frequency cannot adequately cover near and distant parts of the service area simultaneously and at times the 7260 kHz service could not be heard at all at locations up to about 300 km from Port Vila.

However, not all was bad. We had a more or less free hand and a very keen and helpful recipient department of radio communications. In addition we were able to organize the purchase and installation of a used 2.5 MF transmitter so that the 30 year-old veteran could be honourably retired and a breathing space gained.

As a preliminary step in arriving at a total broadcast transmission phase, a comprehensive study was made of the coverage obtained from existing broadcasting stations in the general area. This study took the form of both listening tests and received field intensity measurements.

Measurements of ground conductivity were made of the typical geological formations found on the major islands for correlation with signal strength at MF.

The effect of seasonal and sunspot variations was considered theoretically at this stage and the field intensity measurements continued beyond the design study period as confirmation of assumptions.

It soon became clear that the frequencies being used were not totally satisfactory given the propagation conditions. A country cannot use whatever frequency it chooses. Frequencies are allocated at a World Conference (International Frequency Registration Board and Regional Administrative LM/MF Broadcasting Conference) held at infrequent intervals and small unrepresented countries unfortunately come off second-best compared with the larger nations. However some additional frequencies were available and these were considered in association with those already in use.

Our study report recommended that transmission facilities be installed during Stage I as follows:

Medium frequency serving Efate	10 kW at 1125 kHz
High frequency	2 kW at 3945 kHz (night) 3275 kHz (day) 10 kW at 7260 kHz (day) 3945 kHz (night)

To complement these transmitters and associated control equipment a new 100 m high radiator mast plus a 110 m diameter earth mat was required for the MF service. The 7 MHz HF array had to be upgraded to accept 10 kW power and a new 3 MHz array with associated earth mat.

Completion of this stage would result in good MF coverage to the island of Efate and the capital, Port Vila, and a satisfactory signal in HF at least being received throughout all islands.

Stage II would extend the availability of MF to the larger centres of population outside Efate as follows:

Luganville Santo	5kW at 1179 kHz
Norsoup Malekula	5kW at 1205 kHz
White Sands Tanna	2.5kW at 1722 kHz

Whilst the transmission plan was being prepared, work was also proceeding on the studio facility study. The functional principles were defined as follows:

(a) To establish a facility which would provide an improved broadcasting service for news information and entertainment programmes in English, French and Bislama.

(b) To provide and equip a functional and attractive building with low maintenance and recurrent costs together with a capability for future extension

(c) To liaise with and incorporate the British, French and Condominium's broadcasting and information services plans for utilization and staffing of the station.

All the foreseeable technical needs could be met by providing two transmission/announce booths with associated control room, a production studio and control room, a central equipment room, a record library/dubbing suite and outside broadcast facilities. The administrative areas would include offices for department heads and a large news room.

Accommodation

The two transmission/announce booths were provided to facilitate simultaneous or back-to-back programming of two different language segments. Each booth can accomodate an announcer and up to three interviewees. The transmission control rooms are designed acoustically and operationally so that they can be used as 'on air' announce/control rooms. In this configuration each transmission control room can provide facilities for a single announcer/panel operator or an announcer and panel operator or guest working in the same acoustic space.

Each transmission control room, which is the operational heart of the station, is equipped with a 12-channel audio console having two remote programme inputs, two disc reproducer inputs, two ¼in. reel-to-reel tape reproducer inputs, two cartridge audio tape reproducer inputs and four microphone inputs. Two-way input selection for each channel effectively doubles the number of programme sources which can be accommodated.

It was probable that the Condominium would introduce stereo AM and/or FM broadcasts at some future date. Transmission control room consoles therefore accommodate stereo inputs from discs, reel-to-reel tape and cartridge tape sources. Audio from each remote input and microphone input channel is divided equally between the left and right stereo output channels.

The production studio accommodates up to eight persons seated for panel/interview programmes or approximately 12 persons in a musical/instrumental group. In combination with the control room it can be used for multichannel recording. A high level of sound isolation is provided between this area and the rest of the station.

Within the control room a double-sided control room desk accommodates a 12 input, four output, production audio console with variable equalization on each of the 12 input channels. This console is capable of delivering four outputs into a four channel, four track, ½in. audio recorder/reproducer to facilitate post-production mixing onto a half track mono or two track stereo ¼in. audio tape recorder/reproducer.

Audio programme sources include two disc reproducers, two ¼in. audio tape reproducers, two cartridge audio tape reproducers and six microphone inputs. The console includes provision for pre-fade splits on all inputs plus facilities for studio feedback and reverberation. A graphic equalizer caters for audio special effects.

The control room has been designed with sufficient flexibility to accommodate an announcer with up to two guest interviewees but in the normal production configuration the non-operational side of the desk accommodates the producer and musical director.

The record library dubbing suite has two sound isolated editing booths, each accommodating a disc reproducer, a reel-to-reel tape recorder/reproducer and a cartridge tape recorder/reproducer. A switching panel facilitates dubbing between disc and tape.

The central equipment room is used as the terminal point for all incoming and outgoing programme lines and as the technical communications centre. It accommodates up to six equipment racks, housing equipment such as AGC amplifiers, equalizing line amplifiers, VU meter panels, monitor amplifiers, power supplies and audio jack fields.

A switching system matches output from the studios with input to the transmitters. All output from the transmitters is monitored automatically on a cyclic basis with a programme failure alarm to indicate loss of carrier or modulation on any channel. In addition a slow speed logging recorder is provided to record continuously the output of the studio for monitoring and transcription purposes A talkback main station is situated in the central equipment room to facilitate talkback between the technical areas.

The site made available to us was adjacent to the existing studio facilities and has views over Port Vila Harbour. There was a significant fall across the site which led to the adoption of a part two-storey building. The building has been planned so that the central equipment room forms a nucleus for the technical area and provides an acoustic and environmental buffer between the studio/control room areas and the garage/mechanical plant areas. The entire technical area is located over a services void for wiring and flexibility (see Fig. 3).

Forms of construction

As most buildings in Vila are constructed in reinforced concrete with metal roofs and blockwork walls, it appeared logical to assume these forms of construction were quite familiar to local contractors and are suitable for the seismic and climatic conditions encountered in Vanuatu. This basic philosophy was followed but with the addition of a concrete slab under the metal roof to provide sound and heat insulation and to give total protection during cyclones.

Other external finishes are stucco to blockwork and concrete, with painted lattice sunscreens, in situ concrete eaves guttering and metal storm shutters to all window areas.

The structure of the studio building is a conventional in situ reinforced concrete frame designed to withstand cyclonic winds of 220 km/hr. and earthquake forces in accordance with New Zealand Zone A, in addition to normal live loads. Concrete made from local coral aggregates is satisfactory with a crushing strength of 20 MPa.

The studios and central equipment room are air-conditioned by a central all-air system based on a package-type conditioner. Measures were taken to control structure and airborne noise.

In the event of extended power failure, such as a natural disaster, standby power is available to keep the studio operational. This covers not only the operating electronic equipment and auxiliaries, but also the air-conditioning fans, to remove the heat.

The study report was accepted and Stage I put into effect almost immediately. The transmission facilities were the first to be incorporated and the benefit was immediate with greatly improved services throughout the islands. In fact letters were received from radio enthusiasts in the most unlikely places around the world who had suddenly heard Radio New Hebrides in their earphones.

Studio construction was completed in 12 months with equipment installation and testing taking a further two months. The standard of construction was extremely good, given the general level of expertise available, helped in no small measure by our resident Clerk of Works, Harry Calverley, a long-time Arupian.

With the improved facilities, broadcasting hours have increased to 12 per day and the service is beginning to realize its unifying function. We are proud to have been associated with a project of such importance to Vanuatu.

Associated consultants

Architects:
Ancher Mortlock & Woolley
James Ferrie Associates
(Port Vila)

Services engineers:
Frank Taplin & Partners

Studio engineering & transmitters:
Quantum Electronics

Propagation & aerial design:
L. A. Parker & Associates (Design study)
Noel Medlin (Implementation)

Woden College of Technical and Further Education

Peter Thompson
Graeme Curnick

In 1973 Ove Arup & Partners, in association with architects John Andrews International, were appointed by the National Capital Development Commission to prepare a master plan for the East Woden area of Canberra.

The NCDC proposed an office complex to accommodate 6,000 workers, together with parking for 3,600 vehicles with a pedestrian network linked to the existing Woden Town Centre.

The linear site of approximately 15 ha adjoins the existing Woden Town Centre. The site was subject to flood by the Yarralumla Creek passing through the site. The creek had been canalized into a stone pitched section 7.5 m wide, with uncontrolled floodway either side.

The lined section had been overtopped by floods and fatalities had resulted. The requirement to provide safely for flood flows became a crucial input to the masterplan.

Three basic solutions were investigated:

(i) Provision of an upstream retention basin
(ii) Piping of water beneath site
(iii) Providing an open channel but ensuring adequate overtopping provision and protection for the public.

The open channel solution was adopted as being the most practical and cost-effective.

Alternative routing options were considered and it was decided that the waterway should be channelled through on the most direct route consistent with other planning requirements. This approach suited the architect's concept for the Government offices, envisaged as a series of octagonal buildings, set out in chequerboard fashion, and interconnected at corners with courtyard spaces between (see Fig. 2). However, the open channel could not be allowed to overtop and extend laterally indiscriminately, as this would absorb too great an area.

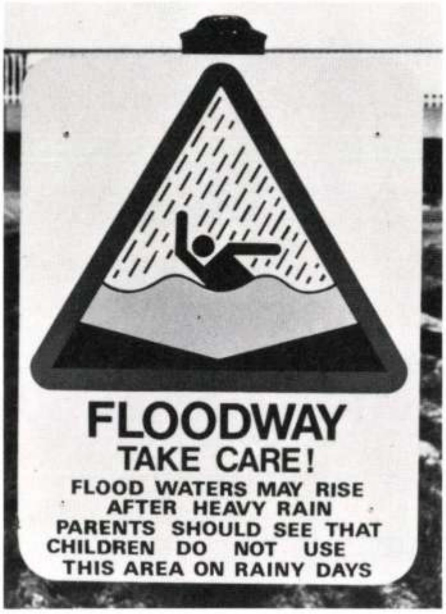


Fig. 1
Flood sign on Woden channel
(Photo: John Burgmann)

In consequence a channel concept was adopted which incorporated a 'hard edge' limiting floodway to one side only and a 'soft edge' allowing emergency exits from channel and floodway.

Estimates of various flood flows were calculated using conventional methods. In addition the concept of a maximum probable flood was introduced. This is a theoretical estimate of maximum runoff from a hypothetical storm of maximum efficiency.

The following quantitative criteria were adopted:

(1) Contain the 100 year flood within lined channel
(2) Contain the 1000 year within a flood plain which is available for landscape and passive recreation
(3) Check maximum probable flood for overtopping the channel and clearance to any structures above.

The accompanying table sets out a range of flow estimates.

Recurrence Frequency (years)	Peak flow (m^3/ sec)
3	40
5	52
10	64
20	78
50	97
100	116
1,000	174
Maximum probable	290

Fig. 3 shows the final channel cross-section with controlled floodway on one side.

To obtain general confirmation of calculated

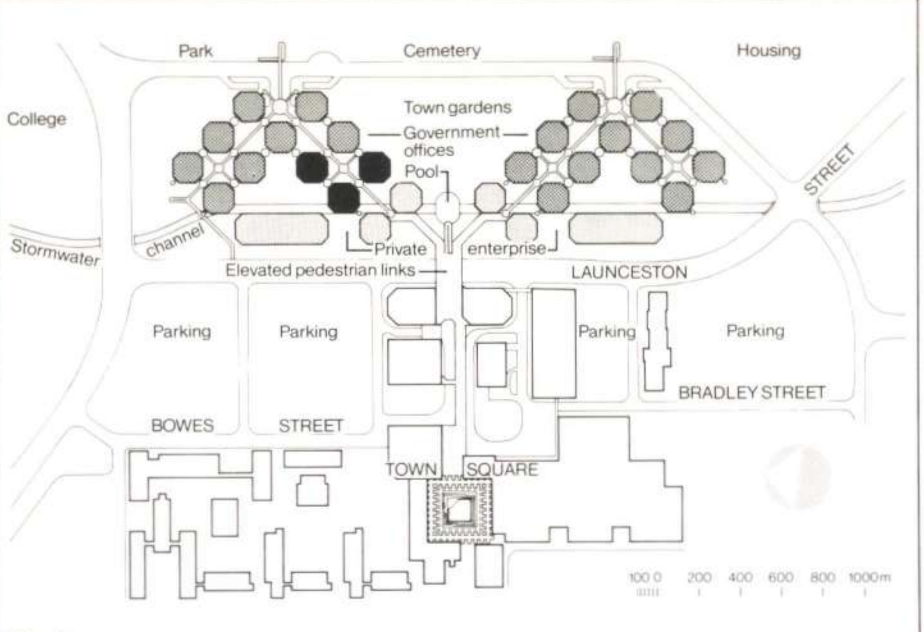


Fig. 2
Master plan

Fig. 5
Space frame sun shading
(Photo: David Moore)

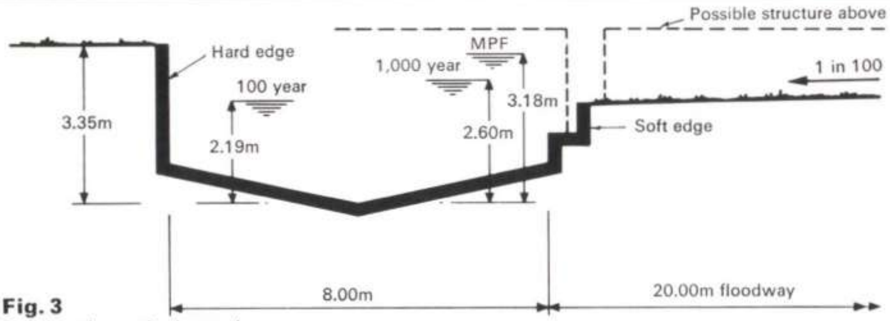


Fig. 3
Section through channel

channel/flood plain performance and to examine wave formation, effects of piers and transitions in plan, a hydraulic model study was carried out.

A concrete model at 1:60 scale represented the full length of channel and floodway to be re-aligned, together with sufficient upstream and downstream length of floodway to ensure satisfactory modelling conditions. The model study generally confirmed the numerical estimates of levels and identified areas of turbulence and wave action in the flow. Testing is shown in Fig. 4.

From the planning exercise had evolved a system of interconnected octagonal buildings located partly within the flood plain and fanning out to the south east and south west. These are identical structures of four levels supported on four columns only so as to minimize obstruction to flood flow. The ground level is open for car parking and the mid level linked to the pedestrian network.

Between the octagonal pods are free-standing towers containing fire stairs, toilets, plant rooms and ducts. Each internal floor is in the form of a deck built off the structural floor, thus forming a continuous underfloor duct space.

To expedite site development a contract for construction of the reinforced concrete channel was let in 1975. By early 1976 the channel was complete so that programmed office construction could commence. However, this was not to be.

Work had proceeded to tender stage in 1975 when the Federal Government changed its policy *vis-à-vis* Government Offices and the project was set aside.

In 1977 it was decided to build three pods for use as the first stage of the Woden College of Technical and Further Education.

This change of usage created little difficulty structurally but modifications were required to the services, and provision for heated and chilled water storage in the form of underground storage tanks was incorporated.

Each octagonal pod is 30 m across providing a space capable of subdivision into small offices whilst allowing a useful penetration of natural light levels and providing occupants with a desirable external aspect. It is connected to its neighbour by a space 12 m wide, the width of one face of the octagon. Four pods in pairs enclose a courtyard.

Each three level pod has four internal columns which support a roof level suspension system comprising post-tensioned, concrete-filled steel tubes. From this, eight perimeter hangers are suspended to support the slab edges. Cross-bracing between the columns above roof level provides increased stiffness to resist unsymmetrical live and dead loads and wind loads.

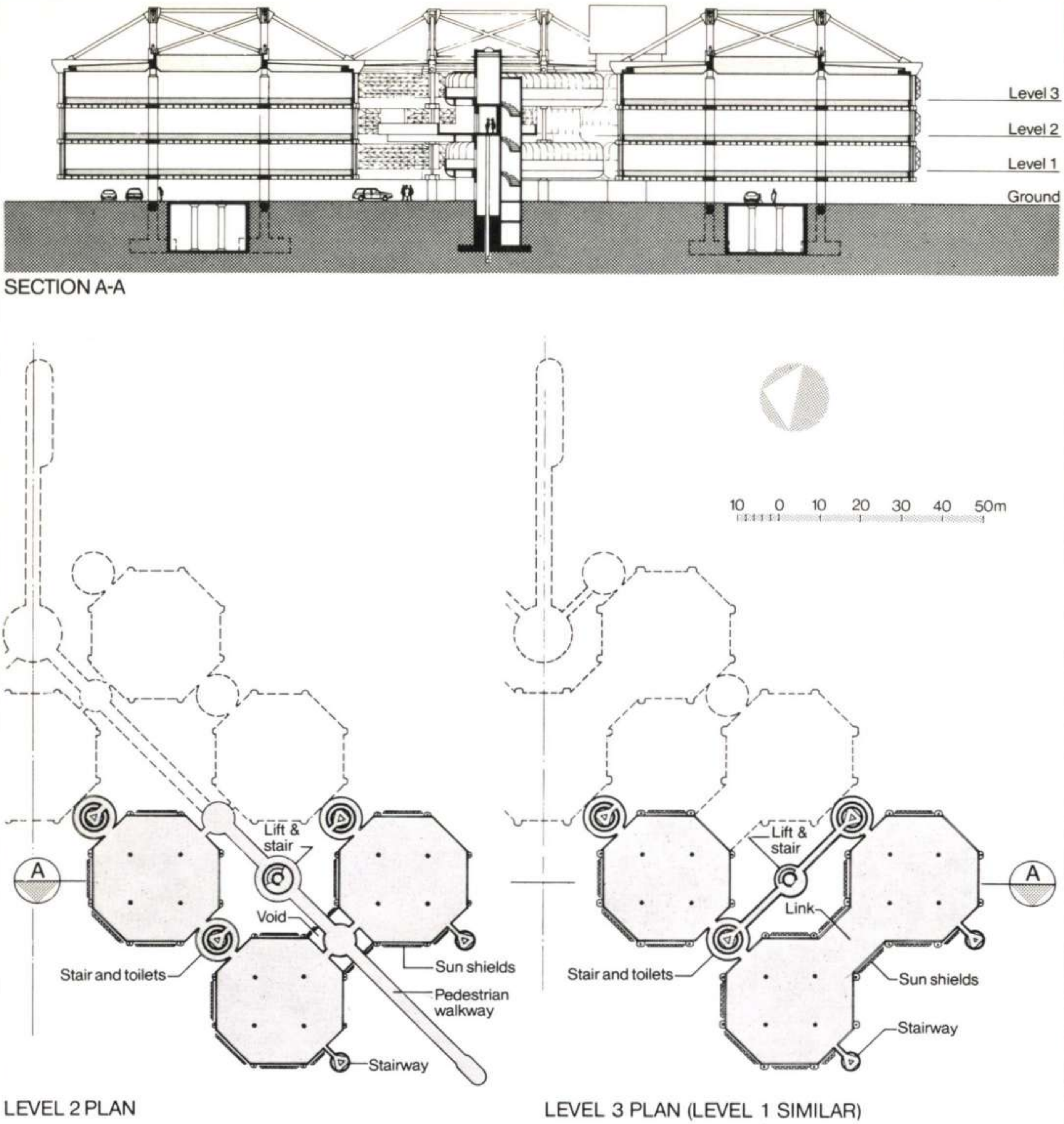
The main tie across the top of the building is a 406 mm diameter tube filled with 40 MPa concrete and is post-tensioned by a single cable containing up to 42 x 12.5 mm diameter seven-wire strands. Cross-bracing members do not contain cables but flat jacks at one end of each brace allow the force to be adjusted.

At the intersection point of the main cables at the top of the columns, a circular hollow steel section was used for the cable duct to minimize the friction loss at this change in direction.

Each perimeter hanger reduces in diameter from 406 mm to 323 mm to 220 mm down the building. Two 12-strand cables are provided in the hangers and pass either side of the main inclined cable in the end block at the eaves level. One cable is anchored by bond in the hanger between level 2 and level 3 and the other anchored at the underside of level 1.

A temporary column was provided from ground level to level 1 and the hangers acted

Fig. 4
Model testing of effect of 1000 year flood on proposed channel
(Photo: National Capital Development Commission)


SECTION A-A

LEVEL 2 PLAN

LEVEL 3 PLAN (LEVEL 1 SIMILAR)

Fig. 6
Plans and section through buildings

as columns for construction of the upper levels. A normal sequence of construction was therefore possible to roof level. The steel tubes for hangers were placed in their final position before filling with 40 MPa or 50 MPa concrete. All of the tubes for the suspension system above roof level were filled with concrete before being placed in position on falsework. The eaves end block and top column junction were then constructed.

Provision was made for flat jacks beneath level 1 hanger point to enable load measurement after each stressing step. In outline, the stressing sequence was:

(1) Stress level 3 to roof cable
(2) Stress inclined tie
(3) Stress level 1 to roof cable
(4) Adjust flat jacks in the crossbracing to a predetermined load
(5) Remove temporary column

Fig. 7
Woden TAFE at night (Photo: David Moore)

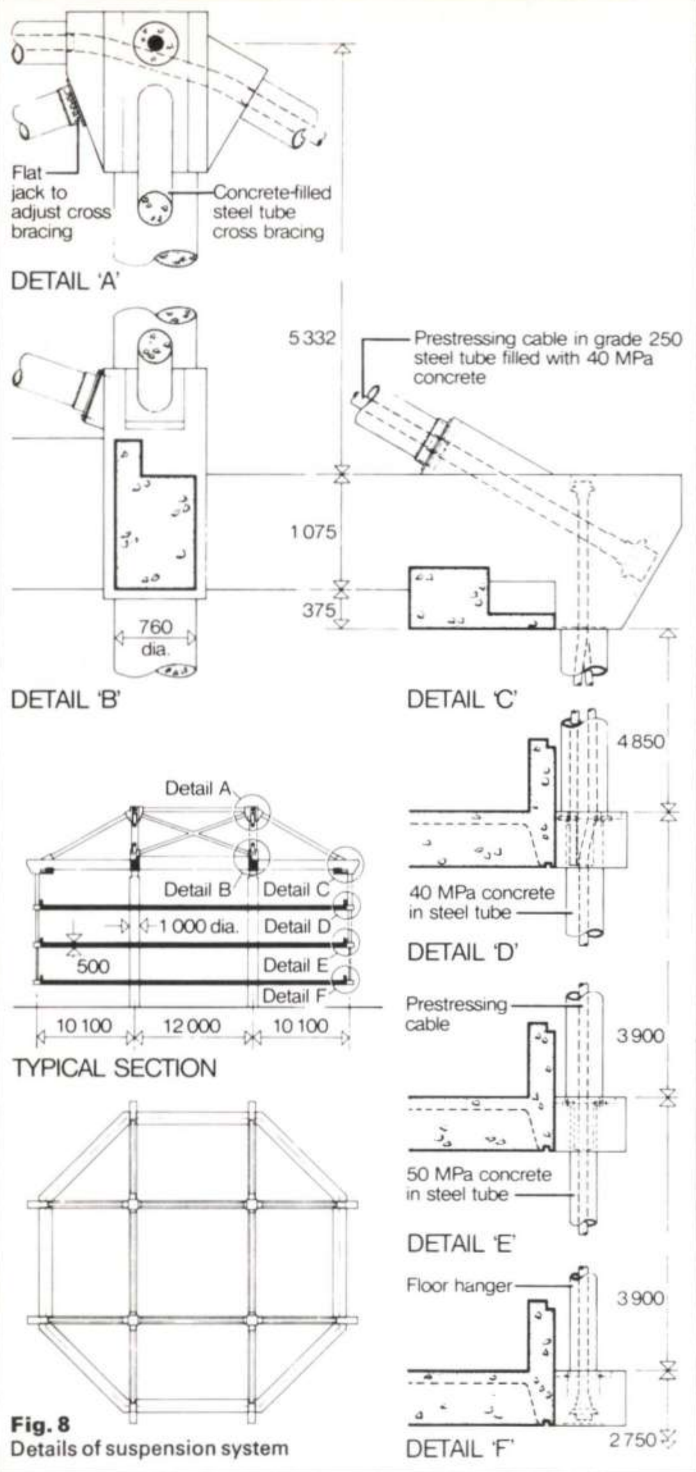
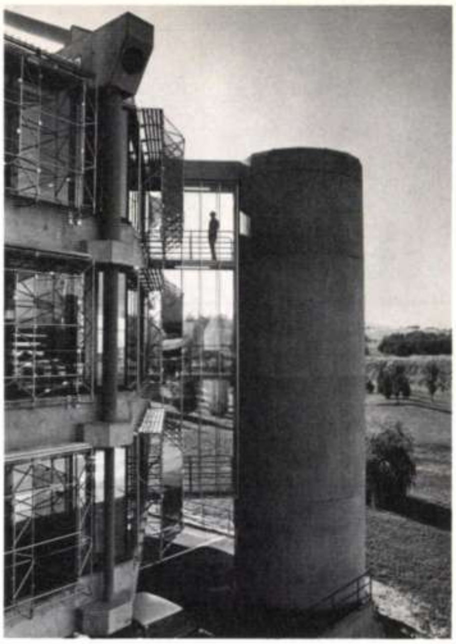


Fig. 8
Details of suspension system

Fig. 9
Freestanding stair shaft
(Photo: David Moore)



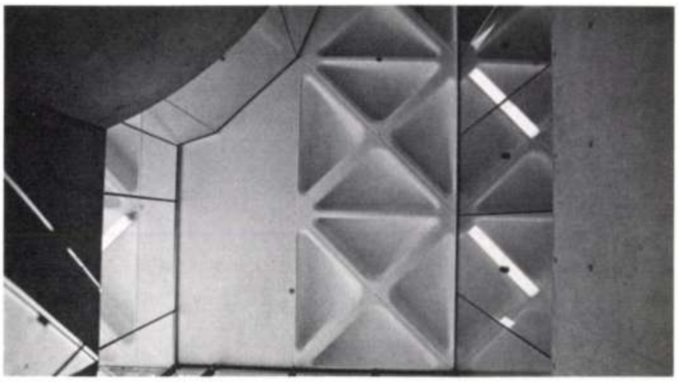
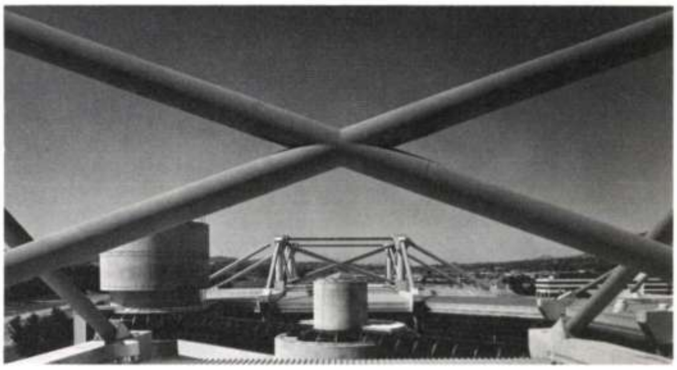


Fig. 10
Waffle ceiling

Fig. 11
Suspension system on roof (Photos: David Moore)



The loads measured by the flat jacks after each step of the main stressing operation were within 20% of predicted loads.

To dispel any fears that the client had regarding the behaviour of the completed structure in service, an outer panel of the level 1 floor was loaded to its design (working) load value. A measured deflection of 4 mm at the hanger point under load agreed with the calculated value. In addition a daily temperature variation, on average, of 0–15°C accounted for a daily movement of about ± 4 mm.

The in situ waffle floor slabs, 0.5 m in depth overall, have structural ribs on a 1.5 m x 1.5 m square grid between a grid beam spanning from hangers. A secondary non-structural diagonal rib forms a triangular pattern.

Purpose-made fibreglass form units were used to cast the waffles which are exposed to form the ceiling finish. Lighting is provided within the pans.

Internal areas are enclosed with full height, double-glazed walls protected from undesirable sunlight penetration by a three-dimensional space-frame fabricated from generally 25 mm diameter stainless steel tubing supporting sun-controlling, toughened glass panels arranged either vertically or horizontally, depending on orientation.

Between the pairs of pods are freestanding towers which also provide vertical access. Around the central concrete core of the stair of each of these towers are glazed passages connecting the office areas in adjoining pods, and, at the first and third levels, enclosed walkways to the central lift.

Concrete underground water storage tanks provide thermal energy storage in the form of chilled water (4.5°C) and heating hot water (50°C) at 200 kPa pressure for use throughout the College as demand dictates.

The structural base of each service tower is formed to provide the chilled water tanks (cooling energy storage). These tanks are interconnected to enable the total storage energy to be made available in any pod via the chilled water, pump-circulated, cooling distribution system provided on individual floors to serve the special needs of the College. A high degree of system flexibility and reliability is provided by this interconnection.

Heating is provided entirely by waste heat utilization by the storage of the rejected heat from the refrigeration process and the reclaim of heat otherwise rejected to atmosphere via the air exhaust from the buildings. The waste heat energy is stored in the two large underground storage tanks and these interconnect with the three refrigeration plants beneath each service tower.

Credits
Client:
National Capital Development Commission
Architect:
John Andrews International Pty. Ltd.
Structural engineers:
Ove Arup & Partners
Mechanical, electrical & hydraulic engineers:
D. S. Thomas
Quantity surveyor:
D. R. Lawson Associates
Main contractor:
Max Cooper & Sons Pty. Ltd.

Power station siting study Queensland

Ron Bergin

Introduction

Power generation in Australia is under the control of the seven State Governments, each operating essentially a separate system. In mainland States the grid is supplied by coal-fired stations backed by hydro-electric generation.

In Queensland the industry is organized into three levels. The State Electricity Commission of Queensland (SECQ) is responsible for generation and transmission planning. They make recommendations to the Government on the siting and capacity of future installations and in turn authorize the Queensland Electricity Generating Board (QEGB) to design and construct the approved capital works project. The QEGB sell in bulk to seven Boards distributed throughout the State who finally sell to the consumer (see Fig. 1).

At the end of 1980 Queensland's generating capacity was 2300 MW with a further 2175 MW under construction at Gladstone (coal-fired), Wivenhoe (pump storage) and Tarong (coal-fired). System growth was 7% and several high demand projects in the aluminium, shale oil and coal mining industries were in the feasibility stage. Lead times for these were less than that needed for a new station.

To meet this demand SECQ's generation planning program included the construction of Callide Station B (2 × 350 MW coal-fired), a Central Queensland Station (4 × 350 MW) to use coal from their Curragh mine near Blackwater and another coal-fired station with capacity ranging between 4 × 350 MW and 4 × 660 MW. The sites for the last two stations were yet to be determined.

Late in November 1980 Ove Arup & Partners with British Electricity International (BEI) were commissioned by the SECQ to act as their consultants and conduct a siting study in three areas, namely Curragh, Rockhampton and Broadmount, for the Central Queensland Station (see Fig. 2).

The terms of reference outlined the study as follows:

'. . . to determine the preferred site for a possible new coal-fired power station in each area taking into account all the pertinent area conditions including topographical and geological information . . . The consultant will carry out economic comparisons of alternative sites in each location to serve as a basis for recommending a site there . . .'

'. . . Consideration should be given to environmental effects but an environmental impact statement is not required . . .'

'. . . A preliminary layout for the power station should be given for the preferred site . . .'

'. . . The final report will contain an estimate of the costs of those components of the power that are directly attributable to the recommended site. These include:

(1) Site preparation and levelling
(2) Foundations for major plant items
(3) Ash and water storages
(4) Ashing and water pipeline systems between the storages and the power station
(5) Coal conveying from the coal delivery point
(6) Provision of roads and services . . .'

The results of this study were to be input into a larger economic study being done by SECQ which would form the basis of their recom-

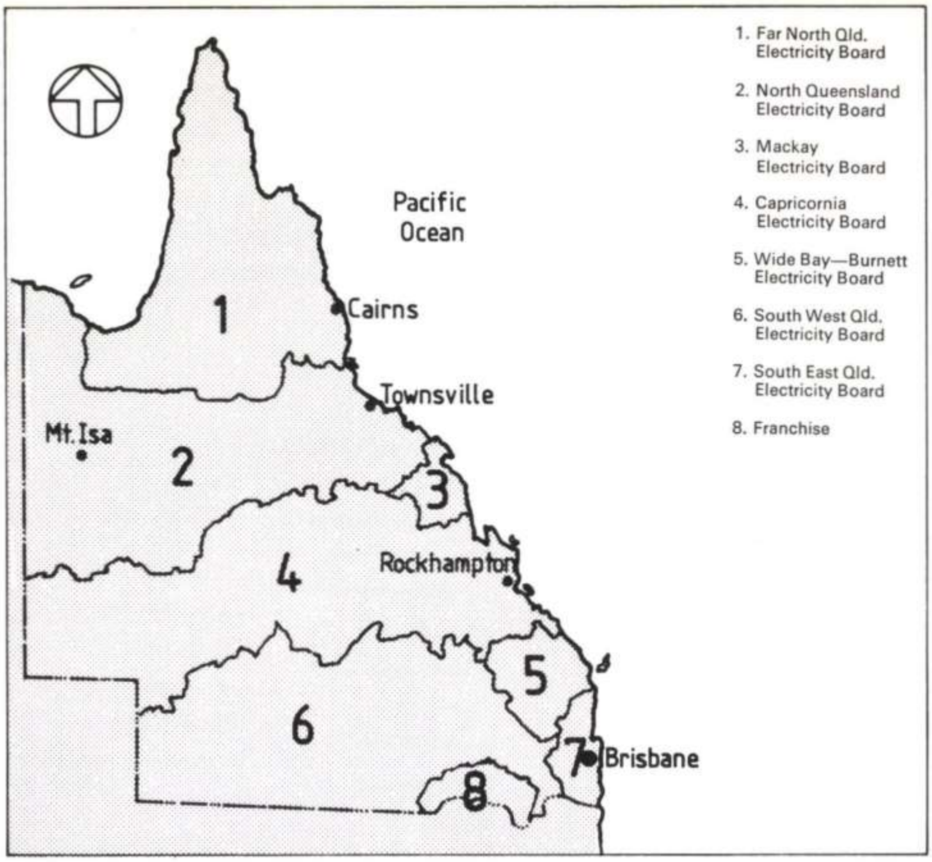


Fig. 1
Queensland electricity boards

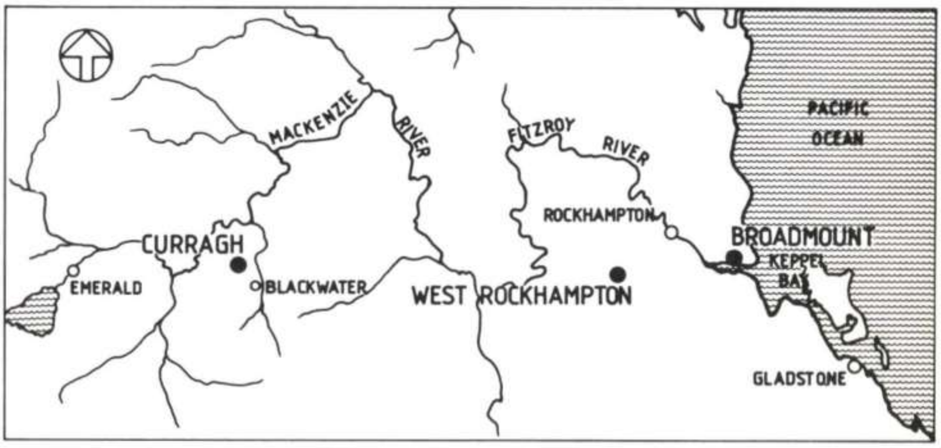
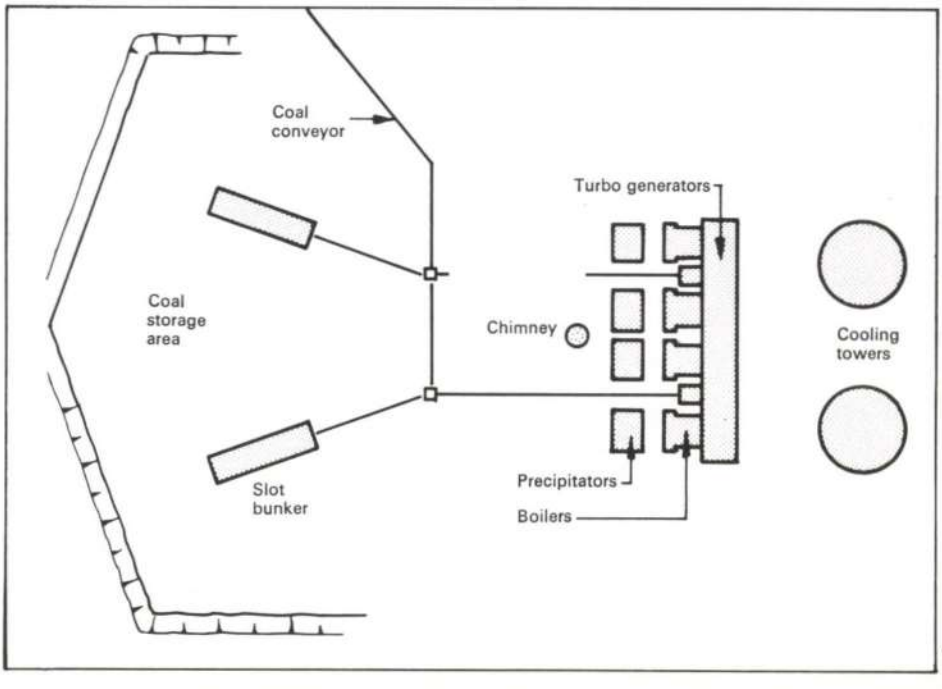


Fig. 2
Study areas in Central Queensland

Fig. 3
Tarong power station layout



4

5

Fig. 4
Curragh

Fig. 5
Rockhampton

Fig. 6
Broadmount

(Photos: John Henry)

6

mendation to Cabinet, scheduled for June 1981, on the location and arrangement for the Central Station.

Arups' study team comprised specialists from the Australian, London and South African practices. Along with BEI they would operate from the Brisbane office. To brief our team the client formed a steering committee comprising SECQ and QEGB members. They would monitor our progress and give interim acceptance or direction on recommendations made in the various stages of reporting. The study was completed in 21 weeks with specific milestones achieved during its course. A report was issued at each milestone.

Central was to be constructed on an expedited program. Unit 1 was scheduled for commission by May 1987, Unit 4 in November 1989. With Callide B and Tarong on accelerated programmes as well, the steering committee adopted a policy for Central of minimum change from the layout principles QEGB had adopted at Tarong (see Fig. 3).

Our planning for Central embraced the following:

(1) A level or terraced area of 70 ha for the main station complex, comprising boilerhouse, turbine hall, cooling towers, precipitators, stack and coal stockpile
(2) An area of 40-70 ha for contractors' storage and fabrication
(3) Capacity to expand the station by 2 × 350 MW sets
(4) A coal stockpile of 1.0m. tonnes, representing three months' supply
(5) On-site water storage of 3.0m. m^3, representing 1 month's demand, for an evaporative cooled station on site
(6) Use of 2 × 700 MW natural draught cooling towers
(7) A once through cooling water (CW) system for a direct cooled station
(8) Use of multi-flue stack
(9) Ash disposal by wet techniques using earth dam walls.

Study areas

Curragh, Rockhampton and Broadmount were distinctly different with a diversity of station planning conditions.

The Curragh study area was defined loosely as the 36 km^2 immediately to the west of the mine and was chosen with the obvious economy of coal transportation in mind.

The surrounding countryside is sparsely settled, flat, arid country used predominantly for beef cattle grazing. Being 200 km inland, meteorological conditions are free of coastal influences. The availability of water, annual rainfall, the net deficit of rainfall to evaporation, and wind directions, would all influence the planning.

Blackwater, a coal mining town of 7000 people, is the residential and commercial centre of the region and would be used to house the constructions and permanent workforce for a station. It is linked by road, rail and air services to the city of Rockhampton on the coast.

The Rockhampton area was chosen as an alternative because it combined rail access for coal supplies with use of an existing city (population 54,000) and its infrastructure to absorb the station's workforce both permanent and temporary. To exploit fully this combination, a larger area was defined for the study. Sites were to be within 15-30 km from the city centre but could lie within the complete semi-circle to the west of the city, a study area of 400 km^2. This region encompasses the rich grazing land of the broad floodplain of the Fitzroy River and gently rolling country, before giving way to the foothills of the Great Dividing Range. Rockhampton is classified a cyclone area.

The third area chosen as an alternative was Broadmount. Broadmount is one of the few sites on the Queensland coast suitable for a direct cooled station. It is located on the north bank of the Fitzroy estuary, 8 km from the mouth on the lower slopes of the ranges before the topography enters desolate salt pan flats and beach deposits. Specifically defined between West Arm Hill and Flat Top Range, the study area was some 8 km^2.

Broadmount today is a remote weekend fishing village with some cattle grazing on the better pastureland. At the turn of the century it was the main port for Rockhampton and linked rail. The line has been pulled up but the embankment can still be seen from the access road which essentially follows its route for the 35 km journey.

Rockhampton would be used as the centre for the workforces of a Broadmount station.

The contrast between the study areas can be seen in Figs. 4, 5 and 6.

Base data

National Map Series 1:100,000 topographic with 20 m contours were available for each area. As the study progressed SECQ arranged for 1:10,000 topographic maps with 3 m formlines to be produced from high level aerial photography. Their accuracy was, however, suspect due to lack of ground control.

Geological mapping was available in the form of a 1:250,000 map series with explanatory notes. Combined with a photogeomorphologist's interpretation of the aerial photography, these were to give the team an understanding of earthworks and foundation conditions likely to be encountered before visiting each site.

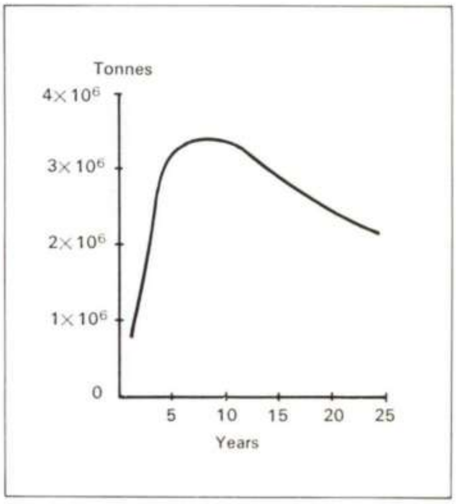
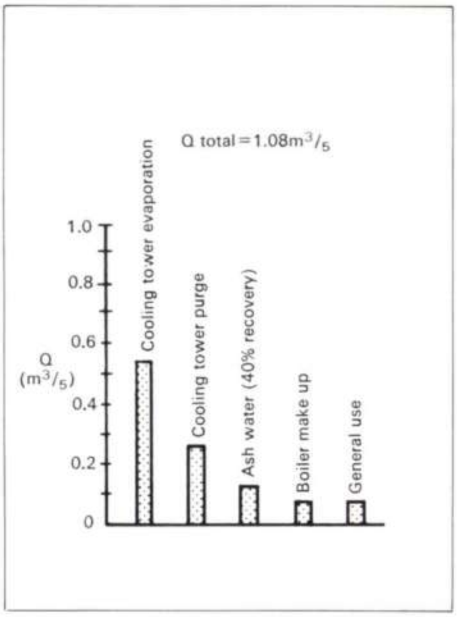


Fig. 7
Estimated coal consumption

Fig. 8
Water demand
(1400 MW station evaporative cooling)



Hydrographic data for the assessment of the thermal capacity of the Fitzroy estuary and the design of the cooling water system included tidal levels, storm surge analysis for cyclones of various return periods, river and estuary channel flows, river silt analysis, coastal drift, sea water temperature recordings and soundings into Keppel Bay.

The load curve for the proposed 1400 MW station was given in terms of estimated annual coal consumption (see Fig. 7). All other load-related consumables, i.e. water, haulage, power, etc., were assumed used in this pattern and their costs assessed in present value terms over a 25-year station life using a discount rate of 8%.

Curragh steaming quality coal was analyzed to have an ash content of 16.5%. Adopting 20% to account for variation and a density of 1200 kg/m^3 in place after hydraulic slurrying, the total volume of ash produced from the 70 m tonnes of coal consumed in the life of the station is approximately 12 million m^3.

Cooling water throughput for the station operating at 1400 MW was determined as 44 m^3/sec. For Broadmount this would be a once through system with water demand at this figure. For Curragh and Rockhampton it would be a closed system with demand determined as 1.08 m^3/sec. A breakdown of this demand is given in Fig. 8.

Cost information was compiled from Arup records or by reference to the construction industry except that:

(1) Water Resources Commission (WRC) would provide costs to supply water from their dams for Curragh and Rockhampton.

(2) Queensland Railways (QR) would provide freight rates for coal.

(3) SECQ would provide transmission costs.

Desk study

Christmas is Australia's summer holiday season and was only four weeks away at commission. We had two alternatives: complete our desk study and field visit to all areas prior to Christmas or postpone a substantial start on the study till January. The latter would effectively lose six weeks and endanger the client's programme to Government. Thus our team was assembled in Brisbane early in December for an intense programme of client briefing, data collection and desk study prior to a site visit to each area with the steering committee.

A Curragh station would be supplied with coal conveyed directly from the mine headworks 3 km to the east. Cooling water would be pumped from the Bedford Weir on the Mackenzie River 12 km to the north and the new mine road from the south would be used for access to the station. Five transmission circuits were required in north, north east, east, south east and west directions.

Topographic maps indicated that local relief was 20 m with no space restrictions for the main station complex. No visual shielding would be provided by the natural landscape. Only two creek valleys were in close proximity, both suited to water storage but only one having sufficient capacity for ashing. Stormwater runoff was to the west and east before entering the Mackenzie upstream and downstream of the Bedford Weir.

Geological maps indicated that the area was underlain by sedimentary rocks of the Upper Permian age, dipping gently to the east at 5°. Mudstones and siltstones of the Burngrove Formation were indicated to outcrop and soil descriptions suggested that quite plastic clays would be found beneath the topsoil.

Stack emissions would generally be carried by the dominant north east-south winds away from Blackwater. However, dust nuisance at the station from the mine was a possibility.

The desk study for Rockhampton was somewhat different, principally because we first had to determine which sub-area of the 400 km^2 had the best economics for station development. By comparison with Curragh it was more a macro-study (see Fig. 9).

Coal supplies would be railed over the Blackwater-Rockhampton line, then by spur line to the site. The longer the spur line, the more economically disadvantaged that site was in comparison.

A similar situation existed with water supplies, road access and transmission.

Water would be pumped from the Water Resources Commission (WRC) barrage in Rockhampton. The greater the distance, the greater the cost, the head differences being less significant. Access road costs would be directly proportional to the distance of the site from the nearest highway while transmission costs were weighted against sites to the north and west away from the load centre.

Topographic maps indicated there were several sub-areas with open space suited to the main station complex in close proximity to natural valleys capable of being dammed for ash disposal and water storage. Drainage of the area was generally towards the east to lagoons on the flats west of the city. The creeks contributing to this system were subject to severe flooding.

Local relief varied within the range 50-150 m and provided varying degrees of shielding from population centres. Whereas winds would generally carry stack emissions westward away from Rockhampton, local effects from terrain increased the risk of the plume grounding at small settlements dotting the countryside.

Published information on the geology of the area indicated a complex mix of soil and rock types with no one sub-area appearing superior to another. A clearer picture would have to await a site visit.

Broadmount was more like the Curragh study. For Broadmount a new rail line would have to be constructed to bypass Rockhampton (the trains currently run through the city streets), bridge the Fitzroy and continue on to the coast. The sea water intake (see Fig. 10) could be located on the north bank of the estuary adjacent to the study area and discharge either downstream or north of Cattle Point directly to the bay, depending upon thermal capacity. A new road would be required from Rockhampton and all transmission would leave the area in a northerly direction to avoid crossing the river and the expanse of mangrove flats to the south.

The main station would be sited in the valley east of Flat Top Range, extending from the shoreline up and over the narrow saddle with West Arm Hill, then opening out to the north and east. Cooling water pumping costs would be a major factor in the economics of the station. To be above the high water of the spring tide and maximum storm surge, the basement level would have to be a minimum of 7 m Australian Height Datum (AHD). The saddle was 10 AHD, thus sites north of the saddle would incur a pumping penalty unless conduits were set below existing ground level.

On the other hand, the available area south of the saddle was confined.

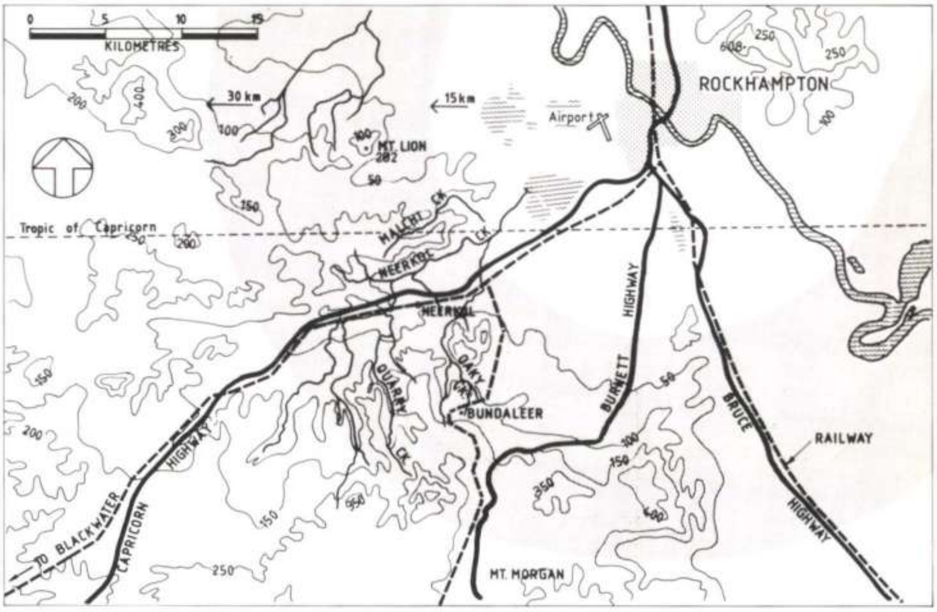


Fig. 9
Rockhampton area

Fig. 10
Broadmount area

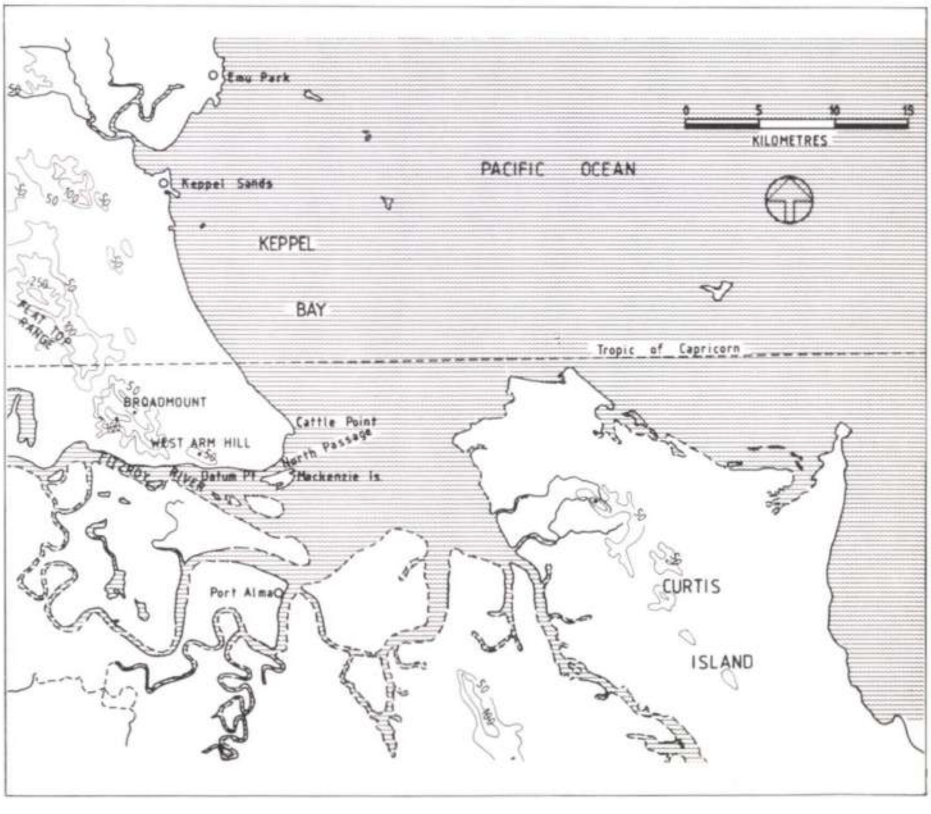


Fig. 11 Site study teams assembling for the day (Photo: Ron Bergin)

The range formed the east-west watershed and though catchments were small, runoff was expected to be rapid with the majority finding its way to salt pan flats east of the station site. These flats were considered suitable for ash disposal behind earth bunds.

The Beserker Beds comprise the Broadmount area. The volcanic and marine rocks of this formation were overlaid by a mix of hillwash, alluvium and beach deposits. Vast areas of salt pan occurred along the northern bank of the river. Foundation conditions here would be very poor.

Due to its remoteness, visual shielding would not be a real issue but the plume could hug the ranges and be carried towards Rockhampton by the prevailing south east wind.

Keppel Bay bed contours were complex. Most of the bay was within the 20 m contour with muddy sand shallows extending quite a distance offshore. The estuary itself had a number of deep channels through otherwise sand shallows. Advective transport of river sediments into the bay was small compared to the nearshore longshore drift north from Cattle Point.

The Fitzroy River has a catchment of 14m. ha. Much of this receives only 600-700 mm of annual rainfall with more than 80% falling in the summer. Flood statistics indicate flows of 10,000 m^3/sec. and 18,000 m^3/sec. at return periods of 10 and 50 years respectively. Due to the width of channel at the estuary, river levels would be marginally increased by floodwaters. Apart from times of flood, river flows were essentially tidal only, with currents and tidal flows in the estuary region moving in a complex pattern not fully understood.

More needs to be known of the flow pattern and sea water temperatures before even an initial assessment of thermal capacity can be made.

With this understanding of the planning requirements and constraints we set off with three members of the steering committee to visit each area.

Site study

The study team was organized into the following groupings:

(1) Power station planning
(2) Water engineering
(3) Civil engineering
(4) Geotechnical engineering.

Our field party of 10 was organized along similar lines with two nominated as official photographers. Each member was issued with a kit containing essential mapping and data. Portable tape recorders were used to put down daily notes. Each evening a project meeting was held to discuss the day's findings and agree plans for the next day. Secretarial services were organized at each centre to make available a hard copy of recorded notes and minutes the next day.

A Queenair and a Beechcraft Baron were chartered to overfly each area before rendezvous-ing with three Toyota 4WD LWB vehicles (see Fig. 11). Once at the site the groups would divide to conduct their specialist assessment and reconvene for discussions later in the day.

Activity hinged around the power station planners. Their assessment embraced space requirements, station layout options, access for rail, road and transmission lines, earthworks and foundation conditions, water storage and ashing and environmental problems. Opinions were naturally sought from the specialists.

The water engineer assessed natural valleys for their capacity and economics to serve as ash disposal dams and water reservoirs, and checked on access for the water supply pipeline.

Earthworks, foundations and constructional aspects of the access road, rail spur line and balloon loop, bunkers, conveyors and stockpile, transmission towers and ash and water dams, were the province of the civil engineering group.

The geotechnical engineers examined the creek beds, cuttings and outcrops to determine the rock types underlying the soils of the area and develop some understanding of the profiles to be expected. They could then advise on likely foundation and earthwork conditions. Sources of aggregates for concrete and roadbase were noted.

For Broadmount all specialists paid particular attention to the difficult conditions surrounding intake and tailrace options of the cooling water system involving construction of the channel either across the salt pan or along the river bank, a distance of 4-6 km. The estuary itself proved as complex as the desk study had indicated. The Science Department of Capricorn Institute of Advanced Education (CIAE) were commissioned to gather data of flow patterns, tidal length and shallow and deep seawater temperatures for an assessment of thermal capacity.

After a hectic six days, the team returned to Brisbane to consolidate findings of the site study and take a well-earned Christmas break. The New Year would see the study unfold with the issue of:

(1) Area ranking report – Rockhampton
(2) Site ranking reports – Curragh, Rockhampton and Broadmount
(3) Final report – Curragh, Rockhampton and Broadmount.

Area ranking Rockhampton

It became evident that the economics of station development in sub-areas of Rockhampton depended upon off-site factors. Given reasonable foundation and earthworks conditions the large variable costs were in:

(1) Spur line and rail haulage of coal
(2) Water supply pipeline and pumping costs
(3) Transmission to load centre
(4) Main access road to the station.

Preliminary engineering designs were done and costed for each. An area called Neerkol was recommended for further study after it showed a 45% and 72% advantage over other areas.

This brought the Rockhampton study in line with Curragh and Broadmount. The outstanding task at each was to recommend the preferred site and undertake preliminary station planning and, after client acceptance, to assess the cost directly attributable to that site and layout.

Site ranking

Curragh solved itself. With the mine to the east and the only ash valley to the west the station had to be in between. The only question was where, and this was compounded by the fact that a fully bunded water reservoir in this vicinity with gravity command over the station appeared more economic than using the only valley available for water storage.

This was almost optimization and would have to wait till later. In the meantime a broad order site investigation would be done to assess foundation and earthworks conditions for this basic layout.

Rockhampton was quite different. Around Neerkol six suitable sites were identified. On each of these a station could be arranged several different ways. These were studied and what was considered the best layout for each site chosen.

A preliminary engineering design was done for each of these and the following variables assessed in terms of capital, running and maintenance costs:

(1) Rail construction and coal haulage
(2) Access and pipeline maintenance roads
(3) Ash dam construction, ash disposal and water reclamation
(4) Transmission to a common point on the grid
(5) Water supply including pipeline and storage reservoir
(6) Earthworks and terracing
(7) Coal conveyors.

Certain layouts had the cost penalty of construction to guarantee immunity against flood waters, while others fronting the highway were arranged to be more aesthetically acceptable but at some cost.

The lowest four were within 11% of one another. We recommended a site near Quarry Creek, not only because it was the lowest in cost, but because it had all the ingredients of being able to support the design and construction of the station to a shortened programme. It was a safe site.

The client accepted this recommendation and again a broad order site investigation was authorized.

Broadmount: BEI's preliminary analysis of thermal capacity for the Fitzroy estuary was encouraging. Indications were that an outfall east of Datum Point in the river or north west of Cattle Point to the bay would work for a station up to 2000 MW. The river option involved damming the North Passage and locating the outfall on the seaward side to avoid recirculation. Thus the layout options for Broadmount were:

(1) Main station either south or north of the saddle

(2) River intake combined with outfall downstream or direct to the bay
(3) Road and rail access along the river following the old line or by skirting north of Broadmount itself.

We developed and costed two combinations and demonstrated the greatest variable cost to be associated with pumping cooling water against head. A station and tailrace kept south of the saddle had advantages but the station layout would have had to be different and occupy less space than Tarong.

The client chose to maintain the Tarong layout, and our study into the final stage was to develop the preferred layout with the station north of the saddle. The site investigation for Broadmount progressed with this in mind.

Site investigations

The site investigation at Curragh was done by Groundtest Pty. Ltd. and Rockhampton and Broadmount by Coffey & Partners. To an Arup brief each produced a factual report of the field work and laboratory testing. Arups supervised field operations and wrote the interpretative reports to be used by the other specialists of our team. These reports were incorporated as a geotechnical section in the reports to the client.

The investigations were directed to finding:

(1) Earthworks conditions at the main station terraces
(2) Foundation conditions under the main station complex
(3) Suitable foundation types satisfying settlement and strength criteria
(4) Conditions at dam sites
(5) Sources of material for dam construction
(6) Sources of concrete aggregate and road stone.

Fig. 12 depicts the profiles assumed from the borelogs with the finally adopted main station terrace levels indicated.

At all areas dams and bunds could be constructed from locally won materials. Broadmount and Rockhampton had adequate sources of concrete aggregate and road stone, but these would have to be imported to the Curragh area.

Preferred layouts

The site investigation results were used to guide us to the preferred layout for each site. The power station planner developed a number of alternative on-site layouts combining power station planning principles with known site constraints. Particular care was given to the search for suitable foundation and earthwork conditions. Each layout was costed and, by an iterative procedure of adjustment and further costing, an economic ranking determined. These were combined with the ash disposal and CW systems similarly investigated and costed by the water engineer to give the basis for selection of the preferred layout.

The final selection was made on economic grounds conditioned by power station planning, construction and environmental considerations. The results are shown on Figs. 13, 14 and 15.

Fig. 13, Curragh, shows the main station oriented about a north-south axis and located to take advantage of the superior economics of a fully bunded water reservoir on the higher ground. The stockpile and stack are located to the north, downwind from the dominant north east-south sector of the wind rose. The 3.5 km separation between station and mine would avoid excessive dust nuisance from this source. Cooling towers are placed to the west of the turbine house to avoid moisture carry-over over the main buildings and beneath the stack plume. Conveyor and ash pipeline lengths are relatively short. Station construction is west to east with contractors area east of the main site.

The site is levelled to 169.5 m AHD (see Fig. 12), thereby removing all expansive clays. Basement slabs sit on exposed weak rock whereas pad footings under heavy loads are taken down to the medium strong rock 2.0-2.5 m below. Ash dam and water reservoir walls are constructed of locally won materials.

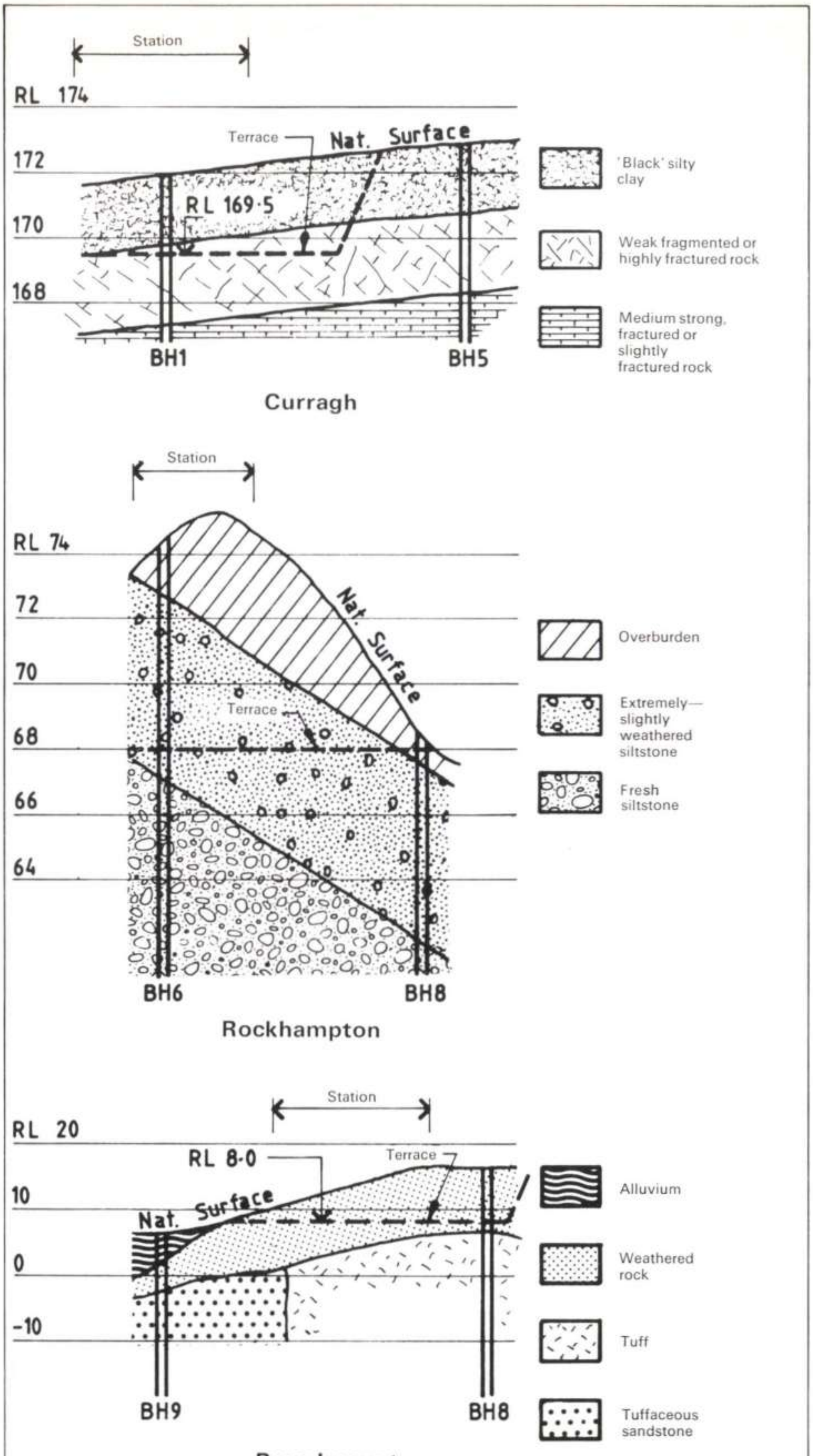


Fig. 12
Borelog profiles

The preferred layout for Rockhampton (see Fig. 14) places the main station directly over the best foundation conditions. With the station elevated and orientated about a north-south axis, the coal stockpile and stack are to the north outside the dominant north east-south east sector. Cooling towers are to the south, minimizing moisture fallout over the station buildings and the influence of tower evaporation on stack plume performance. The balloon loop and conveying system serve the station efficiently from the north.

The existing site is a dome-shaped hill at 76 m AHD (see Fig. 12). By levelling it to 68 m AHD in a cut-to-fill operation all topsoil is

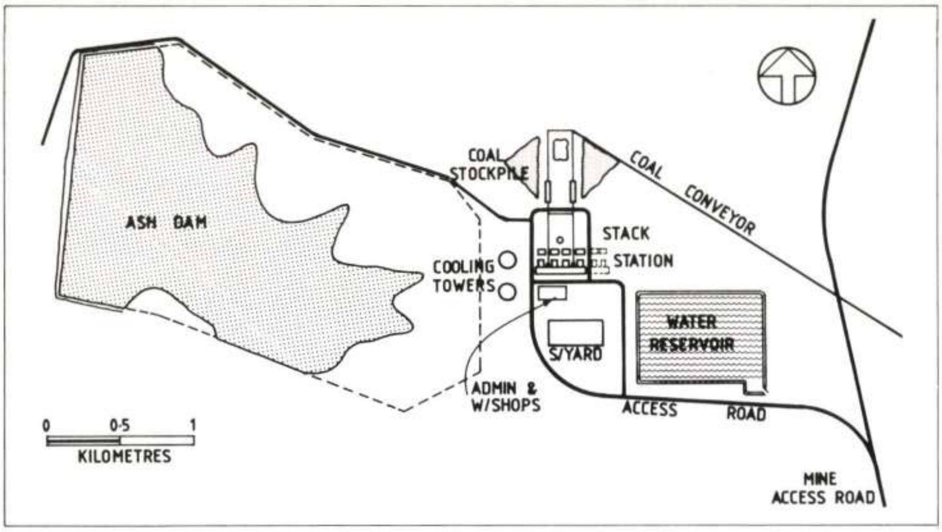


Fig. 13
Curragh

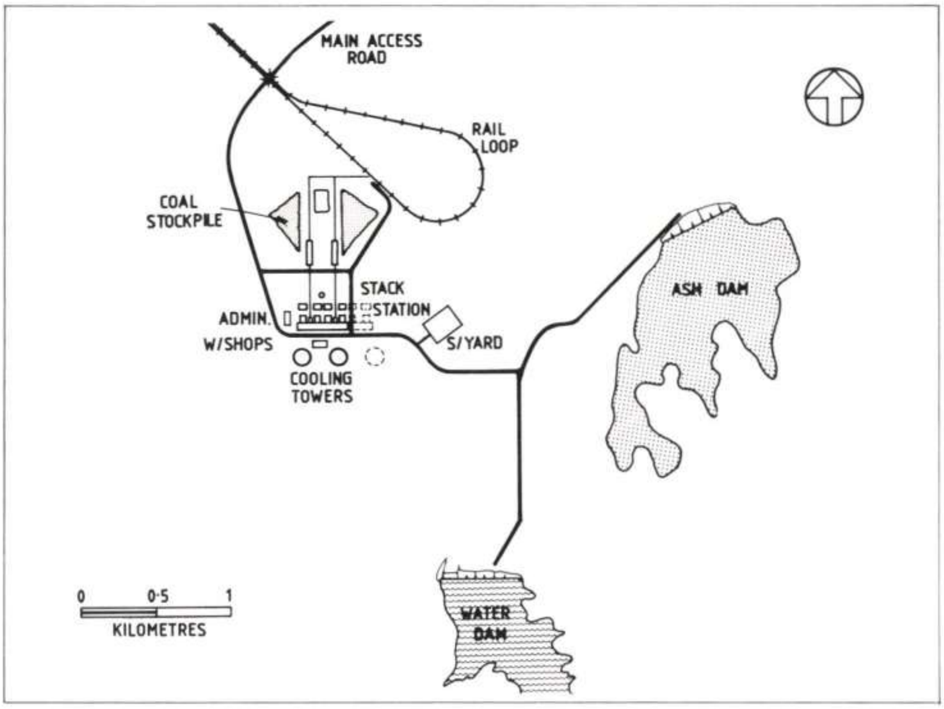


Fig. 14
Rockhampton

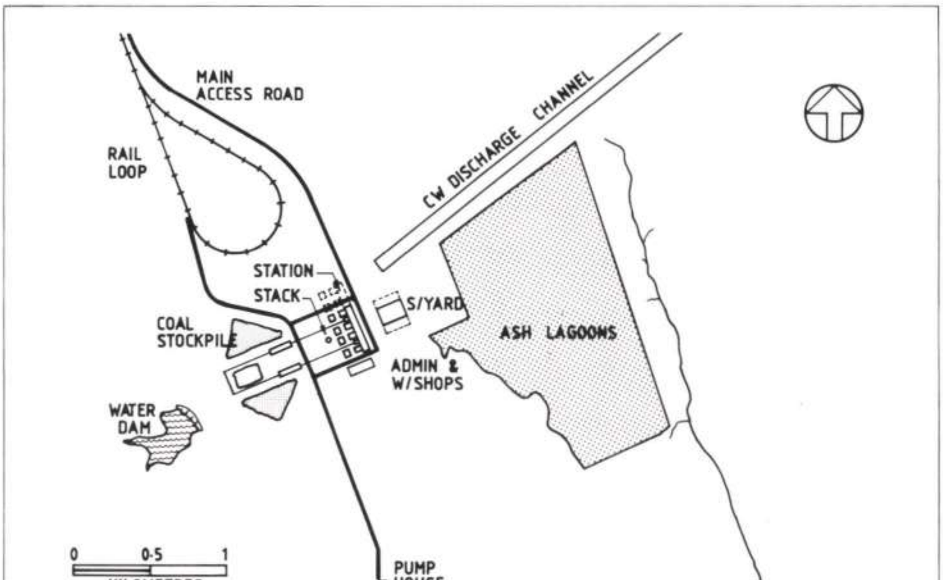


Fig. 15
Broadmount

removed, exposing fresh siltstone in the centre and weathered siltstone and sandstone at the periphery of the main building area. Some rock would not be rippable. Foundations are pads down to the fresh siltstone level. Ground anchors are required to hold down structures in cyclonic wind conditions.

Natural valleys are dammed for water storage and ash disposal. The water dam on Quarry Creek requires a spillway to pass flood waters. The catchment to the ash valley is, however, small and the make (the rain falling within the catchment) is able to be taken into the recovery system. Both dams need a key trench to reduce seepage under the wall but can be constructed from locally won materials.

Broadmount being a direct cooled station was dominated by cooling water system economics. A main station terrace level of 8 AHD provided a practical and economic solution. Thus the preferred layout has the station at this level north of the saddle on the best foundation and earthworks conditions (see Fig. 15).

The station is aligned along natural contours with the station centreline running north east-south west. The coal stockpile and handling provisions are to the west in a sector of low wind frequency. Rail and road access are through the valley from the north. Road and conveyor runs are lengthened to enable sets 5 and 6 to be constructed in a north west direction should the station be extended. Contractors' storage area is to the east of the main site.

The CW system draws its water through a combined intake and pumphouse on the north bank of the Fitzroy and after pumping through the unit, condensers discharge via an open channel tailrace constructed 6 km across salt flats and beach deposits to a sea outfall. The clays and underlying beach sands are of low shear strength in the undrained condition; construction of the tailrace is, therefore, in the form of two connected rock bunds with 3:1 side slopes constructed by an end tipping procedure from locally won material. The salt flats are used for ash disposal by constructing bunds by the same method.

The main station is formed by a cut-to-fill operation exposing weathered rock varying in depth from 3-10 m above diorite and tuffaceous sandstone, both high strength rocks (see Fig. 12). All bulk excavation is considered rippable. Founding is on the high strength rock profile. For depths up to 5 m, pads are used. In excess of this, bored piers are adopted. Again, ground anchors are required for cyclonic wind resistance.

Final reports

For these preferred layouts, the following variables were costed in terms of capital, running and maintenance costs:

(1) Bulk earthworks and terracing
(2) Foundations for all major structures
(3) Main station piping and drainage
(4) Cooling water system
(5) Ash disposal and water recovery system
(6) Ash and water dams
(7) Coal handling system to station transfer towers
(8) Access, station and maintenance roads.

On week 19 the results of our study were submitted to the client as final reports for input into their larger study to recommend the siting and arrangement for the next power station in Queensland.

Credits

Client:
State Electricity Commission of Queensland
Consulting engineer:
Ove Arup and Partners
in conjunction with
British Electricity International

Traffic management: an Australian perspective

David Singleton
Gregory Hodkinson
Glen Holdsworth

Traffic management in its traditional sense has been a component of the urban management system in the developed world for the past 30 years or so; over this period, traffic engineers have been 'managing' traffic in order to squeeze the maximum 'capacity' out of the existing road system, or where necessary, increasing system capacity to provide for traffic demand. Typical objectives of such a management exercise have been to 'provide for the safe, convenient and economic movement of vehicles and pedestrians', with concentration on economy, safety and capacity criteria for private and commercial vehicles. Frequently the needs of other road users (pedestrians, on-road public transport, cyclists) have been ignored and the environmental effects of such 'demand-responsive' traffic management have not been considered.

However, since the early 1970s, more comprehensive approaches to the requirements of urban traffic management have been adopted in North America, Northern Europe and parts of Australasia.

Traffic management exercises undertaken in Australia in the recent past have had a variety of goals:

(1) Reduction of travel times (for all or selected on-road modes)
(2) Improved road safety for all road users
(3) Improved comfort, reliability and security of travel
(4) Amelioration of environmental and sociological effects
(5) Improved economic efficiency of the urban transport system (for the whole community).

Whilst not all the projects that have been implemented have achieved all of these goals, there is an increasing tendency for the majority of these goals to be given consideration in the management process. The needs of all on-road travel modes are generally considered; private vehicles, commercial/freight vehicles, public transport (buses, trams and taxis), emergency vehicles, bicycles and pedestrians, thereby requiring a series of decisions as to which modes should be given priority over others. Community views are now actively canvassed in most road network planning and traffic management projects. A range of levels of public participation in the planning process is entered into, with mixed levels of success.

Regional-scale traffic management

If the traffic management process is considered in a hierarchical sense, then regional-scale road management studies will lie at or near the 'top'. Such studies have been basically concerned with the classification of the roads in an existing road network into a hierarchy, or the development of new road networks in accordance with such an ordered system. Thus the fundamental concept is that lower order roads (local roads, collector roads) feed traffic onto higher order roads (arterial roads) in a twig-and-branch sense, thereby reducing the intrusion of through traffic into residential areas or commercial centres (see Fig. 1).

Whilst the road hierarchy concept is nothing new, having been incorporated into the development of new towns or new residential areas on the fringe of existing towns, i.e. in greenfield situations, its application to areas of existing development has been less prevalent.

In the municipalities of Sutherland and Holroyd, New South Wales (within the Sydney metropolitan area), Ove Arup Transportation Planning have been responsible for the development of road hierarchies or road classification plans for the entire local government area(s). In both cases, this has required that existing roads, generally in a grid-type, non-hierarchical system, be classified according to their desired function and a traffic management plan formulated to reinforce the functional road classifications. For grid-pattern street systems, such management plans are required to be fairly 'brutal', with road closures and other network alterations or the introduction of 'friction' elements required to deny or restrict traffic access to streets now designated local roads. To compensate for the reduction in 'through' traffic capacity which often results, improvements to the arterial road network must be made in conjunction with the local road closures, etc., in order that the overall road transport network continues to function satisfactorily.

Three basic types of traffic management technique have therefore been recommended and applied:

(1) Measures which physically alter the road network
(2) Measures to introduce 'friction' elements to slow traffic and restrict traffic volumes where it is not possible to exclude traffic altogether
(3) Techniques to improve arterial road capacity as a reciprocal measure.

Typical techniques are listed below under these three basic groupings and are also subdivided into physical and regulatory measures (see Fig. 2).

A typical regional scale road classification plan (for the Municipality of Holroyd) is illustrated in Fig. 3.

Physical measures

Network alterations	*Friction elements*	*Arterial improvements*
Road closures	Street alignment alterations	New road construction
Partial road closures	Carriageway narrowing	Grade separation
Conversion of cross to T-junction	Street entrance narrowing	Widening of roads
Diagonal road closures	Off-set intersections	Additional lane marking
Median closures	Roundabouts	Introduction of medians
	Speed humps	Lane channelizations
	Street surface texture	

Regulatory measures

Network alterations	*Friction elements*	*Arterial improvements*
Banned and restricted turns	Traffic signal phasing	Traffic signal phasing
One-way streets	Stop and give-way signs	Priority routes
Heavy vehicle bans	Speed restrictions	Clearways
		'Tidal-flow' lanes

Fig. 2
Typical traffic management techniques

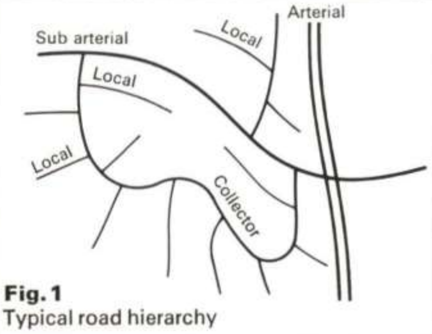


Fig. 1
Typical road hierarchy

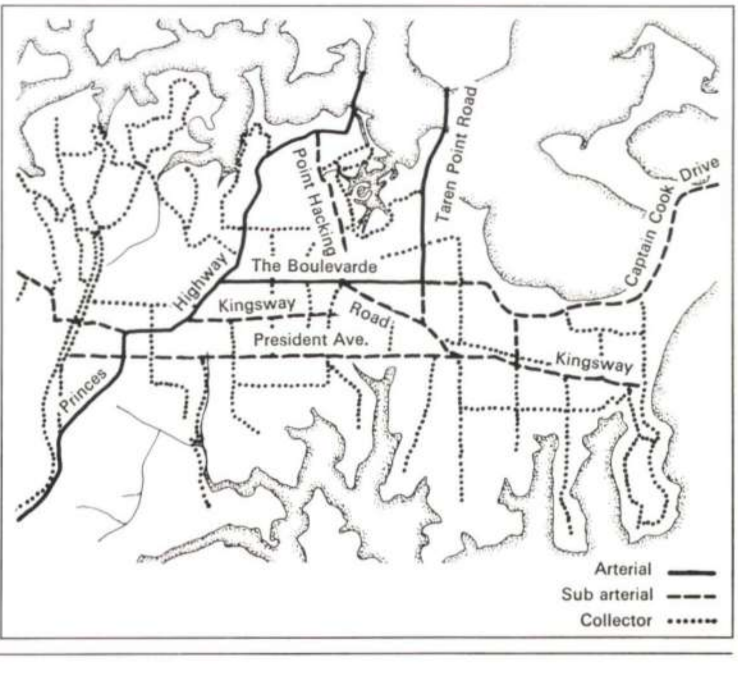


Fig. 3
Proposed Holroyd road classification plan

Environmental capacity

During the development of a road hierarchy plan, consideration must be given to the likely environmental effects of particular traffic routing decisions. Similarly, in a grid-pattern existing street network, a selection will have to be made as to the most suitable road to act as a collector or sub-arterial road. Whilst traffic volumes have traditionally been used in a crude way to measure the environmental impacts of traffic, these alone cannot indicate vehicle types, speeds, times or conditions of flow or satisfactorily measure the resultant impacts of noise, fumes, vibration, danger, intrusion or inconvenience.

Consequently the need for a more detailed measure of the environmental effects of road traffic was identified and Buchanan, in *Traffic in Towns,* proposed the concept of the *environmental capacity* of a street in the early 1960s. Various other researchers have taken the concept further. Ove Arup Transportation Planning (with Planning Collaborative) have been responsible for the development of a methodology for the calculation of environmental capacity in Australia. We have applied it to a pilot study area in the inner suburbs of Melbourne (cities of Collingwood and Fitzroy).

Environmental capacity has been loosely defined as the capacity of a street, corridor or area to accommodate moving and stationary vehicles having regard to the need to maintain environmental standards. More specifically, we have defined the environmental capacity of a street as 'the maximum number of vehicles (and associated 50th percentile speed and proportion of heavy vehicles) that may pass along the street in a certain time period and under fixed conditions without causing environmental detriment'. In the methodology we developed, environmental capacity is calculated in accordance with two primary criteria: (a) traffic noise and (b) effects on the pedestrian environment. In the absence of definitive research as to acceptable situations with respect to traffic noise levels and impacts on the pedestrian environment in Australia, two levels of the criterion variables ('critical' and 'desirable') have been adopted based on standards commonly used elsewhere. Hence for a particular street, environmental capacity values may be calculated for the two criteria (noise and pedestrian environment) for both a 'critical' and 'desirable' situation.

The environmental capacity of a street is a function of the physical characteristics of the street, the nature of the abutting land-uses and the likely speeds and vehicular composition of traffic movements on the street; it is not related to the existing traffic volumes on the street or the latter's function within the road network. Environmental capacity values are calculated following the measurement of determination of a number of characteristics of the street:

(1) Land use (abutting and in general area)

(2) Building setback

(3) Height difference between carriageway and reception point (normally windows)

(4) Carriageway cross-section

(5) Type of ground surface between carriageway and reception point

(6) Carriageway gradient

(7) Presence of buildings at reception point

(8) Types and intensity of pedestrian activity generators in the vicinity.

For any particular street or section of street, the minimum environmental capacity value can be selected, corresponding to either the noise or pedestrian environment criterion, at both 'critical' and 'desirable' levels; the mapping of these values in broad ranges provides a good indication of the sensitivity of an area to traffic-related environmental effects.

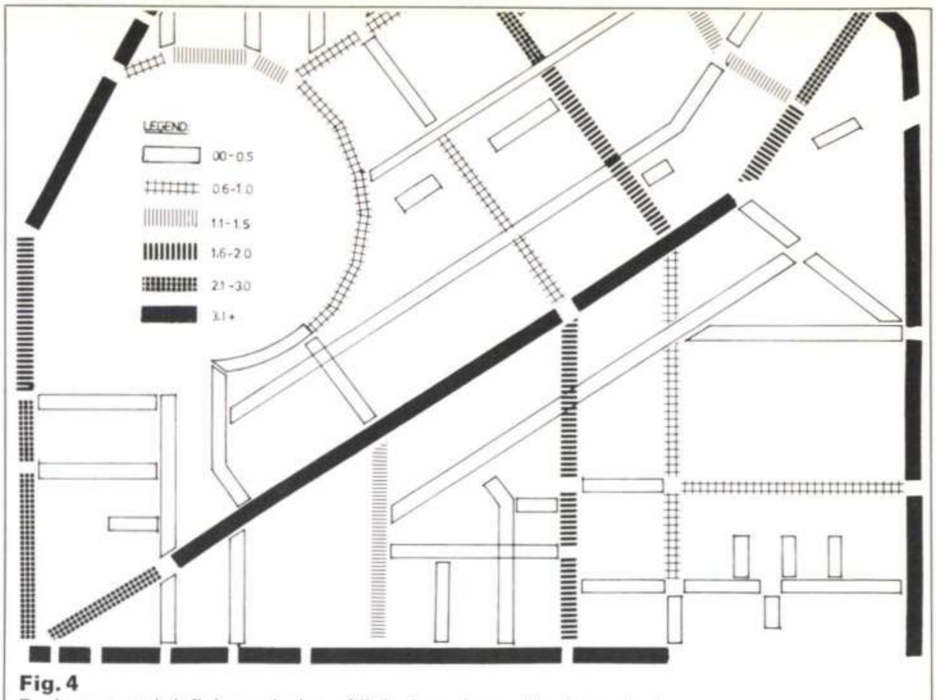


Fig. 4
Environmental deficiency index of links based on critical standards

Fig. 5
Rerouting traffic (Photo: Ove Arup & Partners)

By calculating a factor which we have called the Environmental Deficiency Index (the ratio of current traffic volume on a street to the street's environmental capacity), measure of the environmental state currently prevailing on the street may be obtained. If the Index value is less than one, then 'no' environmental detriment exists; if greater than one, then some environmental degradation is occurring, increasing in severity with increase in the value of the Index (see Fig. 4).

Using the environmental capacity procedure in this way, the concept may be used to:

(1) Determine environmentally acceptable levels and conditions of traffic flow

(2) Identify locations where environmental overload due to traffic is occurring, the cause(s) of such overload and the relative magnitudes of the overload at different locations

(3) Assess potential traffic management measures to deal with identified problems with respect to environmental criteria and select appropriate ameliorative action

(4) Rank locations requiring traffic management/ameliorative actions in terms of environmental need.

Hence the concept as developed in Australia will provide a useful technical aid to the traffic manager, which can be used to identify a need for traffic management or to contribute to a fuller understanding of the consequences of proposed management actions, such as the choice of a street to act as a collector or sub-arterial road. Further research is required to determine acceptable levels of the various criteria and to clarify the perception of such effects by the community and the trade-offs that they are prepared to make. Some work is continuing on this topic in New South Wales by other researchers.

Local area traffic management

The completion of a road classification scheme for a region or single municipality (as described earlier) will provide the opportunity for the management of traffic within local areas so as to preserve or improve the local environment. By virtue of the systematic nature of the road classification process, areas bounded by roads of arterial or collector status may be designated local traffic areas or precincts. Traffic management measures can then be implemented to remove traffic other than that having an origin or destination within the area (see Fig. 5).

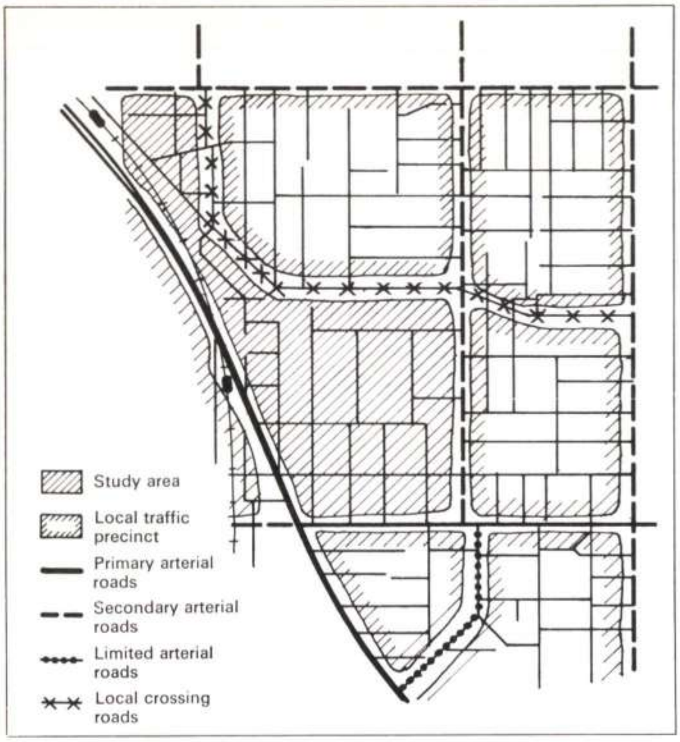
This seemingly simple and straightforward task, when applied to an existing developed area, is in fact possibly one of the most complex a traffic manager will face in his career, as the development of an implementable plan that is responsive to local issues and desires is extremely difficult and at times very frustrating. The two main causes are:

(1) Local traffic management measures interfere with the individual's personal territory and accessibility.

(2) Local community and political issues cannot be separated from the issues of local traffic management and the manager is drawn (willingly or more usually unwillingly) into the political process.

Hence, although the techniques which the traffic manager will propose to employ are tried and tested, their application will generally require the sensitive application of practical engineering skills and extended liaison with the community. Ove Arup Transportation Planning have undertaken a number

6



7

8

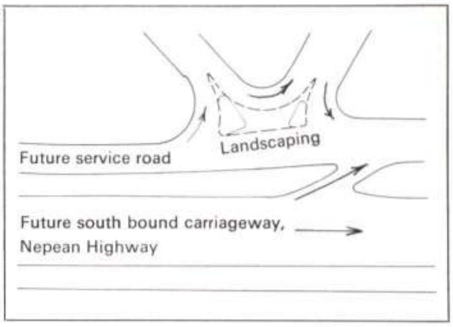


Fig. 6
Gardenvale Study Area management strategy

Fig. 7
Typical trial of proposed management strategy (Photo: Ove Arup & Partners)

Fig. 8
Nepean Highway: St. James Parade treatment

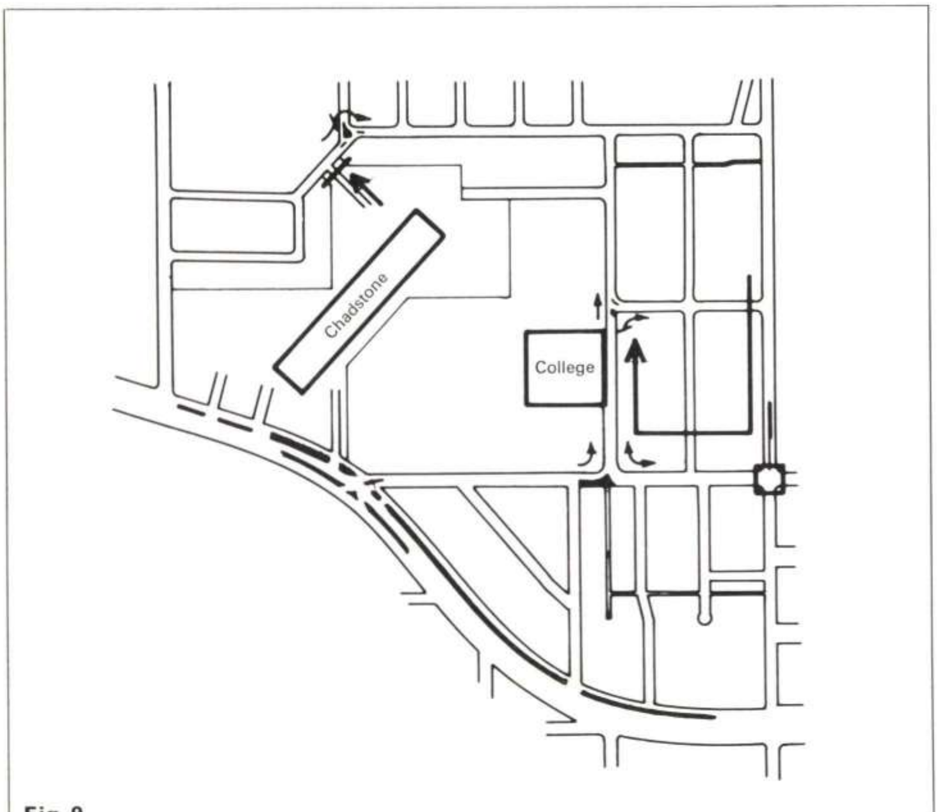


Fig. 9
Chadstone proposed traffic management action

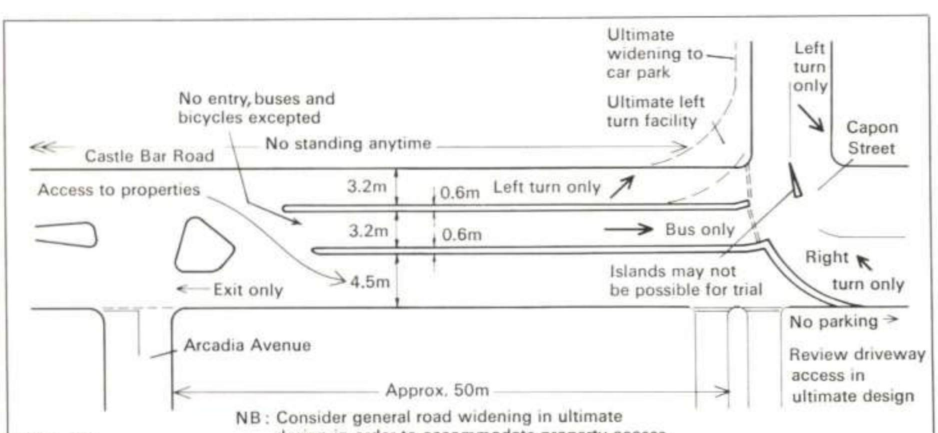


Fig. 10
Castle Bar Road/Capon Street recommendations

of these studies in Victoria and New South Wales recently, with projects for the cities of Caulfield (Gardenvale Neighbourhood Traffic Study) and Malvern (Chadstone Traffic Management Study) in Victoria, providing good examples.

The Gardenvale Study Area is part of a middle-ring residential suburb, also containing two schools, a major hospital and (on the fringe) an extensive motor car showroom, used car lot and repair workshops development. The study area formed a local traffic precinct within a local traffic area (the latter bounded by arterial roads and traversed by collector roads, illustrated here) and hence traffic planning considerations had to extend beyond the boundaries of the designated study area (see Fig. 6).

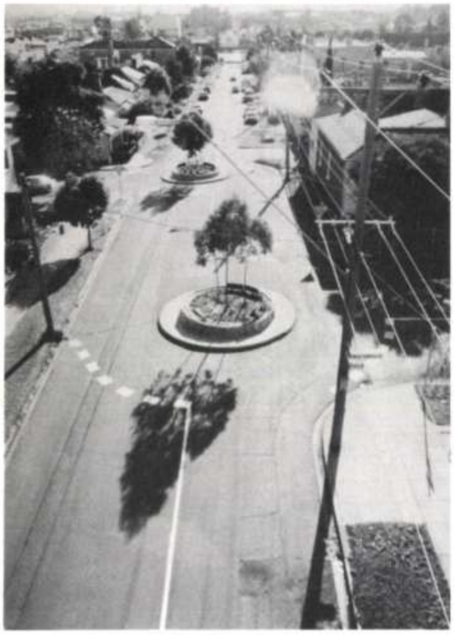
The area was known to suffer from a through traffic problem and the treatment of this problem, which manifested itself in a variety of locations, was a major objective of the study. Observations of traffic movements, including origin/destination surveys to determine through traffic proportions, were carried out and the intensities of these movements formed one of the bases for management decisions.

Concurrently with this quantitative assessment of traffic issues and problems, extensive canvassing of resident opinions (and those of the operators of the various facilities located within the study area) was carried out, by means of 'door-knocks' and pre-arranged interviews. Once planning proposals had been formulated, key resident groups were re-approached to inform them of the proposed management strategy. Recommended treatments were installed on a trial basis and are currently being permanently implemented; a typical channelization treatment control for traffic access/egress to the arterial road on the area boundary is illustrated (see Figs. 7 and 8).

A similar study for the Chadstone neighbourhood in Malvern concerned a residential area dominated by a major regional shopping centre (35,000 m^2 gross leasable area (GLA)) constructed in the late 1960s. Also contained within the study area are a large convent, a teachers' college and two schools. As a result of the presence of the shopping centre, numerous bus routes traverse the study area in order to gain access to the centre. Arterial roads bounding the area were the subject of extensive detailed re-planning to incorporate

11

12



13

Fig. 11
Road closure and parking

Fig. 12
Dual roundabouts

Fig. 13
Pavement narrowing at T-junction creating protected parking

(Photos: City of Prahran, Victoria)

the city-end terminus of a freeway bringing traffic in from the south-eastern and southern suburbs of Melbourne. Consequently, the Council identified the need to develop a traffic management strategy for the neighbourhood as an input to the arterial road management proposals.

Various data collection tasks were performed and the extent of the existing through traffic problem and likely future through traffic scenarios identified. The current and likely future significance of the shopping centre as a generator of trips having a trip-end within the local traffic precinct (notionally legitimate traffic) was also analyzed.

In parallel with the procedure, in a similar public involvement campaign to that undertaken in Gardenvale, advertising in the local press asked for submissions from the public, and following the development of proposals, various public meetings were organized by the Council to inform residents of the recommended management strategies (see Fig. 9).

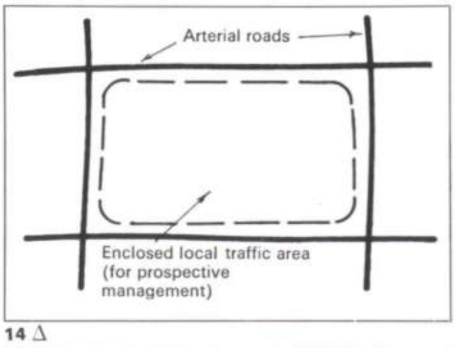
The recommended management actions are illustrated for the study area and several of the shorter-term actions are being implemented at present (see Fig. 10). A proposed special treatment of a T-intersection on what is currently a significant through traffic route within the area is illustrated; this treatment is designed to preserve west to east through bus access whilst eliminating general through traffic movements and maintaining access to properties.

Bicycle planning

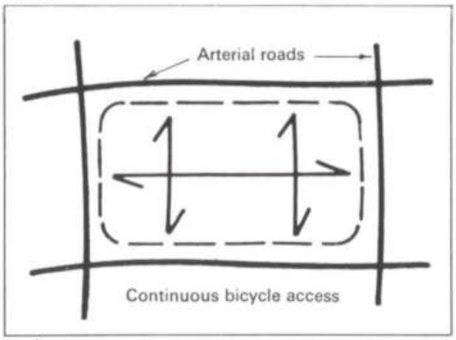
In common with other countries of the world, Australia is currently experiencing a substantial increase in the use of cycles for work-trip and recreational travel, and this has prompted extensive attention to bicycles in the transport planning and traffic management process. At the same time, there has been a realization that a large cycling group exists (and had existed for many years) within the community, namely school-children in their early teens. Various researchers have shown that cycling by this group represents in excess of two-thirds of total trip-making by bicycle.

Ove Arup Transportation Planning has been and is currently involved in various bicycle planning studies for municipalities preparing their own 'bike plans' (Werribee Shire, Shire of Sutherland and city of Malvern). Perhaps more importantly, we are reviewing road traffic regulations and laws as they refer to bicycles and cycling and we are preparing a 'Planning and design manual for bicycles' for use by municipalities for the State Bike Committee, Victoria. We have also planned provision for bicycles into the majority of regional and local area traffic management projects recently undertaken, as part of considering all modes when formulating traffic management proposals.

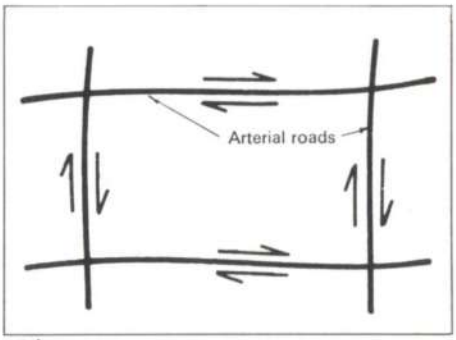
Bicycle planning work in Australia has generally concentrated on providing for bicycles as a legitimate transport mode, and hence user, of the road network, with traffic management measures being implemented to provide for the safe use of the local road network by bicycles. Where bicycle use of arterial routes is necessary, separate bike lanes within or adjacent to the carriageway have been considered; however major emphasis has been on managing traffic such that bicycles may use existing roads, without the necessity for expensive construction of separate rights of way. It can be seen that the basic objective of traffic management in local areas, that is the removal of through traffic, will serve at the same time to create a safe environment for cycling. If links between adjacent local traffic areas (across arterial roads) can be developed, then the opportunity exists to develop regional-scale bicycle routes, paralleling arterial traffic routes but using lowly trafficked roads. However, given



14 △



15 △ 16 ▽



17 △

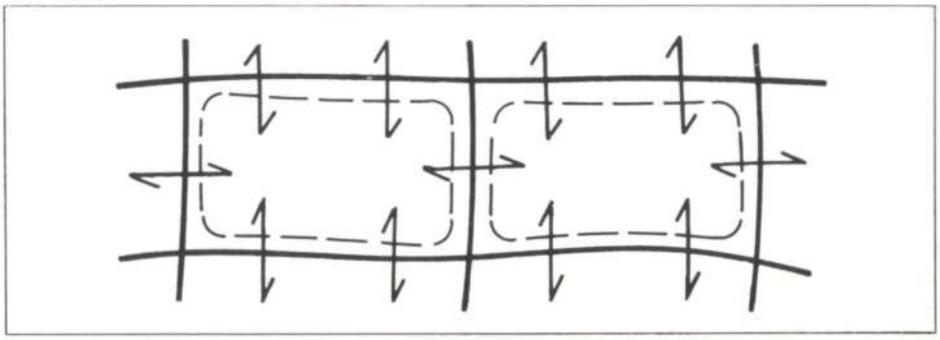


Fig. 14
Establish local traffic areas

Fig. 15
Establish bicycle access within local traffic areas

Fig. 16
Establish bicycle continuity between adjacent local traffic areas

Fig. 17
Establish bicycle traffic integration and provision on arterial roads

the current median trip length for cycle trips (2.5 km), the prime emphasis in providing for bicycle traffic is (correctly) aimed at local traffic areas and hence is well-served by current traffic management practice. The bicycle planning methodology utilized by Ove Arup Transportation Planning in various studies is summarized below:

(1) Establish local traffic areas (see Fig. 14)

Local traffic areas are defined as those areas within the arterial road network. The arterial road network is defined by a road classification (or road hierarchy) planning process.

(2) Establish bicycle access within local traffic areas (see Fig. 15)

Maintain and improve access within local traffic areas (possibly in conjunction with the restriction of unsuitable motor traffic). Ensure continuity through road closures, across railways, creeks and other physical barriers, through street discontinuities, at street disorientations, along and across open spaces and about hills.

(3) Establish bicycle continuity between adjacent local traffic areas (see Fig. 16)

Particular attention should be paid to the relative street orientation and discontinuities across local traffic areas, and to activities which concentrate bicycle traffic, and hence 'desire lines' for arterial crossings.

(4) Establish bicycle traffic integration and provision on arterial roads (see Fig. 17)

Provision for cyclists on the arterial network by various treatments, the suitability of which will be dependent on road classification and physical features; more specifically upon road width, gradient and surfacing; traffic management infrastructure (intersection control, etc.); traffic use and behaviour; and land use.

Where treatment is required to facilitate bicycle traffic on major roads, a range of options aimed at providing for cycling within the existing carriageway is possible and is indicated here, as is an illustration of a successful bike/parking facility in operation:

(1) Shared bike/parking lane (see Fig. 18)

Practical treatment for collector roads

(2) Wide kerb lane (see Fig. 19)

Ideal treatment for collector roads
Practical treatment for sub-arterial roads

(3) Shoulder delineation (see Fig. 20)

Ideal treatment for busier collector roads
Practical treatment for sub-arterial roads and arterial roads in certain circumstances.

The bicycle mode, long neglected in the transport planning process in Australia, as elsewhere, is now receiving due recognition, both as a practical transport alternative for the future and as the only real means of transport for a large section of the community. The development of design guidelines for the provision of facilities for bicycles is a significant and continuing task for the transport engineer and one in which Ove Arup Transportation Planning is closely involved.

The science of traffic management is now well-developed and in many respects, practices in Australia are at the forefront of those adopted in the western world. Through the activities of the firm in Victoria and New South Wales, Ove Arup Transportation Planning has made a positive contribution to the development of these techniques and is regularly involved in their application.

Through the development of the environmental capacity methodology for assessing the environmental effects of road traffic, Ove Arup Transportation Planning has provided a quantitative basis for many of the difficult decisions which must be made concerning the routing of traffic. By the formulation of an engineering approach to planning for bicycle travel and through active campaigning for the consideration of the bicycle as a legitimate transport mode and as such to be given due recognition in the traffic management process, Ove Arup Transportation Planning has assisted in providing for bicycle travel within a safe but achievable framework. It is hoped that these kinds of contribution to the urban and transport planning will continue as the firm maintains its involvement in this sphere of consulting activity.

Fig. 18
Shared bike/parking lane

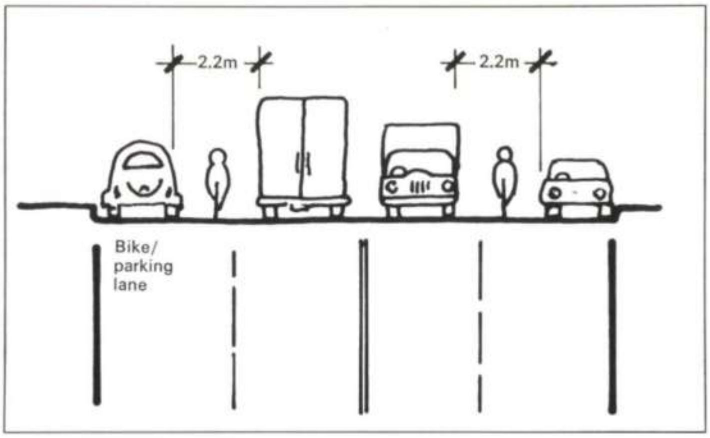


Fig. 19
Wide kerb lane

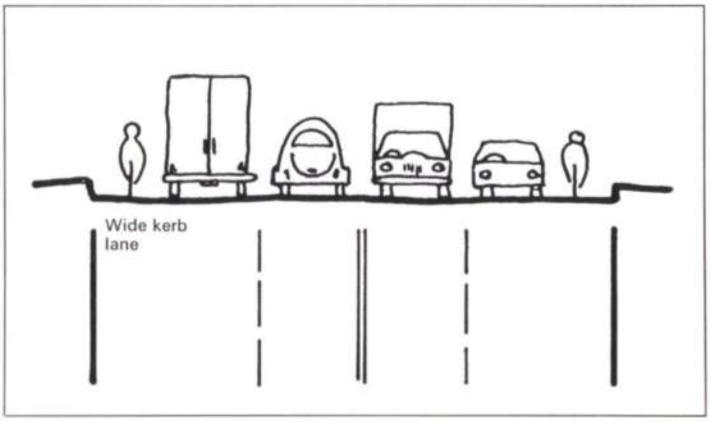


Fig. 20
Shoulder delineation

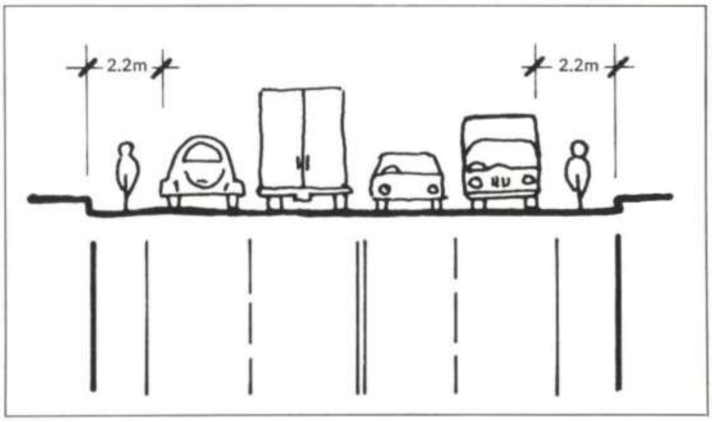


Fig. 21
Bike/parking lane
(Photo:
Ove Arup & Partners)

The CBA Centre, Sydney

Ian MacKenzie

The CBA Centre is located in the business sector of the City of Sydney. Being close to major transport facilities the site extends the pedestrian network of the strategic plan for the city. The building is positioned on a 0.5 ha T-shaped site with three street frontages.

The brief from the Commercial Bank of Australia (CBA) required a development to accommodate the state head office and to provide significant office space to be available for letting. It was thus important to achieve separate identities for the banking facilities and for the office development.

The project consists of a 38-level building, comprising a tower of 28 office floors and three plant room levels rising above a series of plazas. In addition to a banking chamber and other ancillary facilities appropriate to the state head office of the CBA, the seven podium levels provide car parking and several levels for retail shopping arcades. The banking chamber, which is flanked by an internal succession of landscaped mezzanine levels, is a large irregular shaped area with a major glass façade to two street frontages. An underground link gives direct pedestrian access to the Wynyard Concourse, a major transport facility in the city (see Fig. 3).

The typical tower floor is an unusual six-sided shape, formed by curtailing the apexes of a 60° triangle. A central Y-shaped core contains the lift banks and, at the midpoint of each truncated axis, a large hollow column is utilized as a riser duct for vertical services. The windows of alternate floors are set back from the perimeter, giving alternate large and small floor areas and resulting in a distinctive indented appearance.

Design philosophy

The tower is of composite construction. Comparative studies between concrete and steel had shown similar direct costs. However, the construction time evaluated for the steel scheme was significantly shorter and, when translated into costs, tipped the scales in favour of a structural steel solution for horizontal members. The major vertical elements are of concrete which is particularly economic for compression members.

Having decided to adopt a steel structure on the basis of speedy construction it was important to ensure that inhibiting factors were not built in to the design. Careful consideration was given to the questions of layout and design approach as well as to detailing. The latter were particularly important since a connection which was difficult to construct could well have been repeated many times in the course of 28 essentially similar floors.

The design was made as flexible as practicable to allow the builder the opportunity to examine a variety of construction programmes. Tenderers were advised of the range of possibilities in the tender documents. For example, potentially critical areas such as the two high level plant rooms and the lift motor room could, if necessary, be constructed independently of the typical floors.

The second fundamental decision concerned the manner of connecting the structural steel floor beams to both the concrete core and the concrete service shafts. Provision of a pocket or recess in the concrete walls had a basic simplicity which was at first appealing. Relatively straightforward support systems could have been devised which would have catered for tolerance and restraint requirements. However, instances where an opening existed immediately below a beam would have required special details. In addition the beams would sometimes have penetrated the core walls at positions where there was a heavy concentration of reinforcement. It was also felt instinctively that it was preferable to present a steel erector with a familiar steel-to-steel connection rather than an unfamiliar steel-to-concrete arrangement. The seating pocket was therefore discarded in favour of a steel plate cast into the wall and to which seating cleats were subsequently field-welded.

In tall buildings, adverse climatic conditions can restrict available crane time. It was therefore desirable to minimize the weight and quantity of material to be transported to each floor level. This led to the selection of 110 mm thick concrete floors cast on permanent structural steel decking, *Bondek*, acting compositely with the concrete topping. The steel support beams were placed at close centres to avoid the need for temporary propping of the deck while the concrete topping was being placed (see Fig. 5).

The structure up to level 8 comprises essentially conventional reinforced concrete flat slabs or beam-and-slab construction as required to suit the planning. Stage-stressed, post-tensioned concrete beams provide clear spans over a basement cinema.

Wind loading on the tower is resisted by concrete core walls forming the three lift banks. They were coupled together by means of heavy lintel beams. A 1:400 scale model was used in a wind tunnel investigation to establish design wind loads. The model was elastically restrained and took into account the mass distribution and the frequencies of the lowest translation modes of vibration of the building. Surrounding buildings were included in block outline to reproduce turbulence characteristics of the site.

Bending moments were determined at the base of the model and it was established that the loading could be adequately represented by an equivalent trapezoidal static distribution. Wind-induced accelerations at the roof

Fig. 1
Sydney skyline taken from the west (Photo: John Nutt)

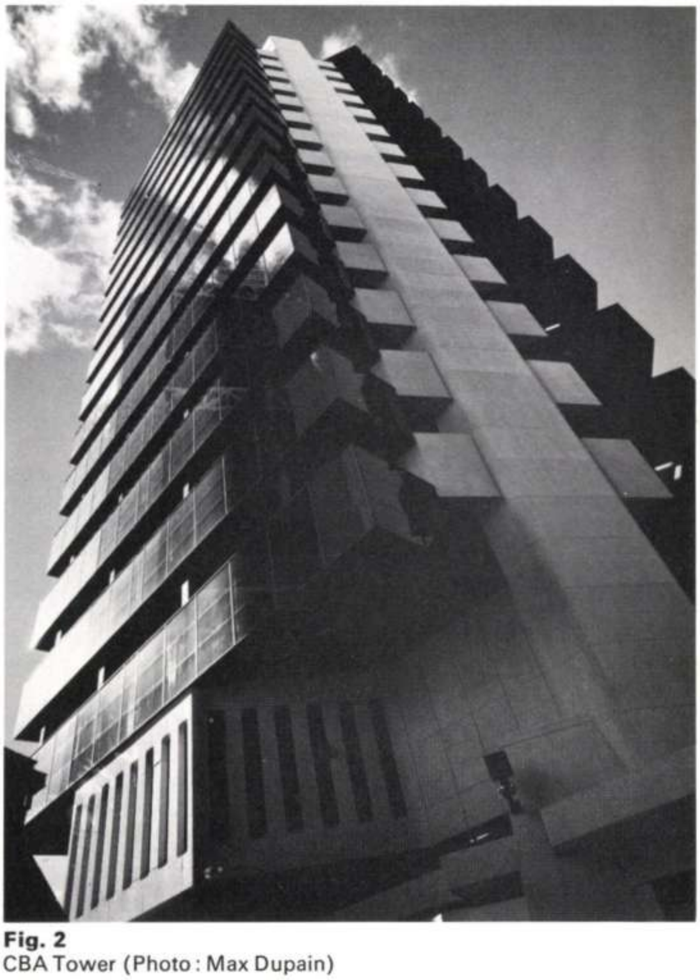


Fig. 2
CBA Tower (Photo: Max Dupain)

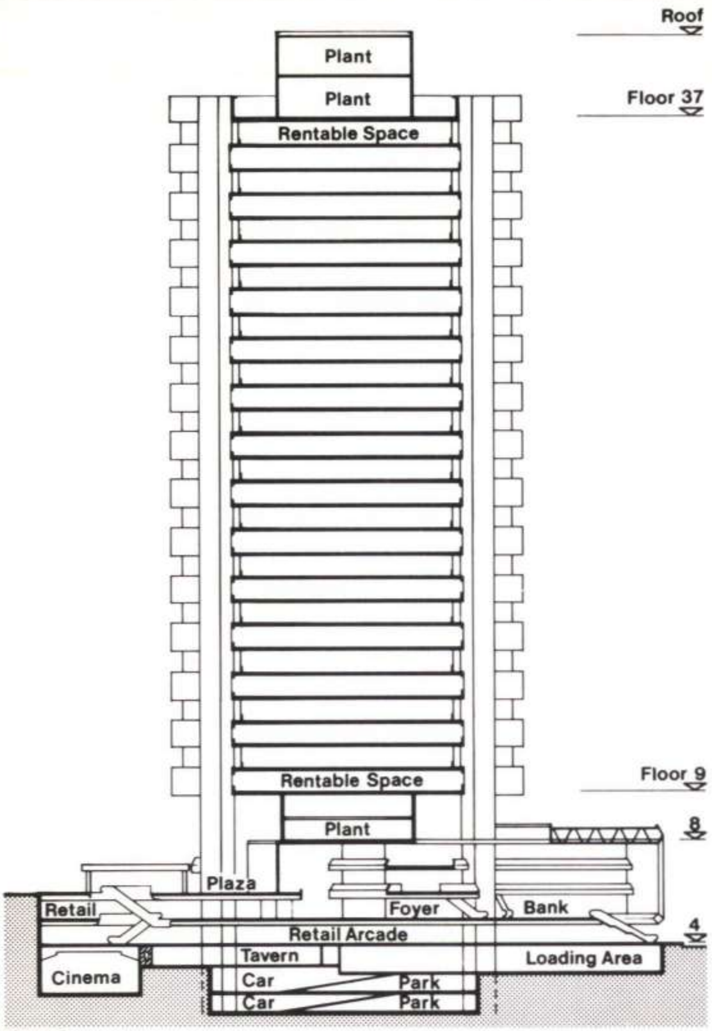


Fig. 3
Section

Fig. 5
Typical floor framing plan

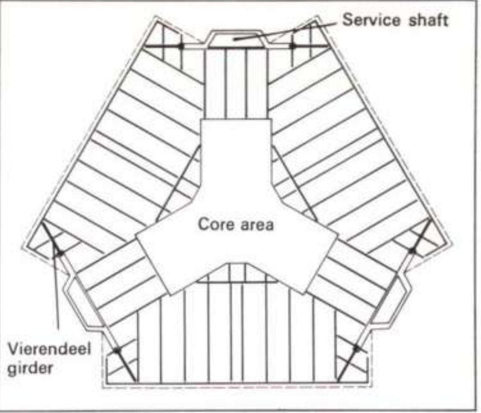


Fig. 6
Tower under construction (Photo: John Hearder Pty. Ltd.)

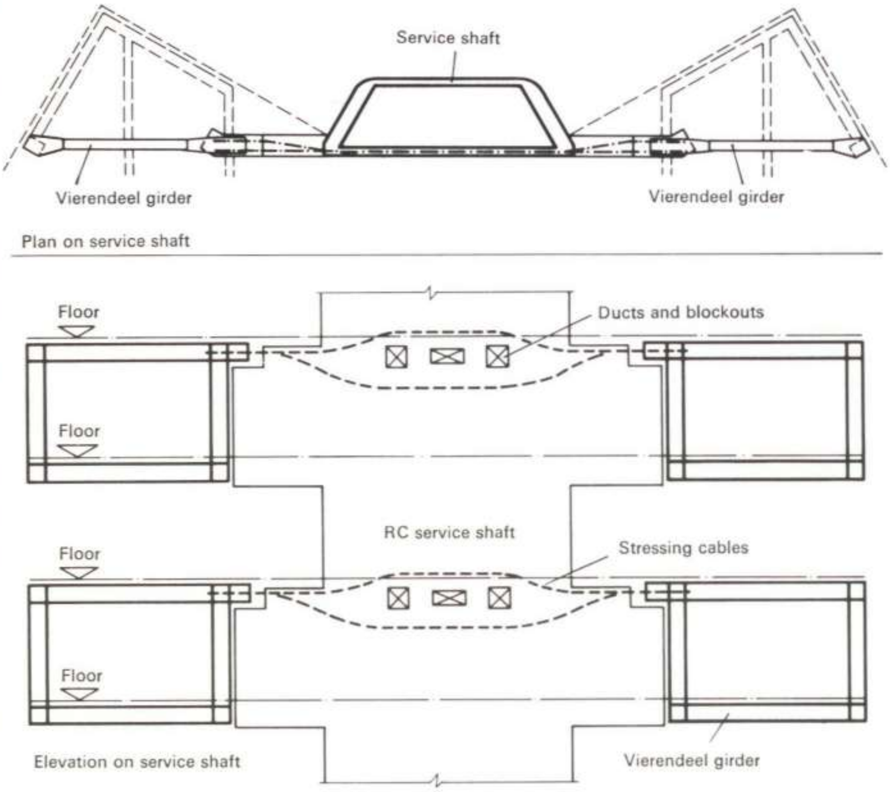


Fig. 4
Plan and cross-section of vierendeel girders

of the model were also studied and the results showed that the accelerations were within acceptable limits.

Each typical tower floor consists of composite steel deck and concrete slab, supported on steel beams which span up to 12.5 m between the perimeter and the core to give column-free space. In addition, support is provided to pairs of columns on each of the three principal façades, by storey-height vierendeel girders which cantilever from the corner service shafts (see Fig. 4). The majority of the floor beams are universal sections to *AS 1204*, Grade 250, but it was found more economical to use Grade 350 steel for all fabricated sections. The load factor method of design was adopted, advantage being taken of composite action wherever practicable. The resulting structure was relatively flexible and, since the requirement for temporary propping of both beams and deck had been eliminated deliberately, all longer-span beams were precambered to avoid excessive sagging in the final condition.

Fire protection of the metal deck soffit was not necessary but, since the deck acted as the main slab reinforcement, a $1\frac{1}{2}$ hour fire resistance rating of the floor structure was achieved by the use of draped supplementary reinforcement in the floor slab. The steel beams were sprayed to contour with a gypsum vermiculite plaster.

It was anticipated that the lift cores would be constructed using slip-form techniques and, as noted previously, it had been decided to adopt seating cleats welded to cast-in plates to provide support at concrete walls. Slipformed concrete cannot be constructed to the tolerances which are expected for steel erection and connections were designed which would allow for significant degrees of variation. First the plate to be cast in was oversized in height and width by 100 mm. In addition, it was specified that the stopping level of each floor-to-floor slip should be at approximately the level of the underside of floor beams, to enable the plates to be positioned in the wall while the slipform was static, thus minimizing the possibility of misalignment.

To overcome the effects of errors in the plan location of the wall, the seating cleats were designed to allow for the load to be applied with an eccentricity of ± 25 mm. The need to design for the moments resulting from the extremes of these tolerances made both the cleats and the cast-in plates substantial members. However, the end result justified the approach as only one instance was recorded in over 1000 connections where field alterations were required.

The structural form of a vierendeel girder is unusual in the context of repetitive floors in a multi-storey building. Each single-panel girder cantilevers 5 m from a concrete wall which in turn cantilevers 3 m from the concrete service shafts. As a storey-height frame, the top and bottom booms are concealed within the ceiling space of the successive floors, while the external posts appear visually as a column on alternate floors only. Members are fabricated as I-sections except for the external post which is a hollow box shape. The restrictions on the available depth for the members led to the use of up to 40 mm thick plates for the typical floor girders while non-standard girders associated with plantroom levels used plates ranging up to 50 mm thick.

Connections of booms to the outer posts were complicated, not only because the I-section translated to a box profile, but also since the axis of the post was oriented at an angle of 30° to the plane of the girder. Careful planning, including the use of full-size polystyrene models, enabled the fabricator to confirm assembly procedures and weld preparation techniques.

The most complex junction was that where the girders were restrained to the concrete wall. At this location anchorage plates were required for the post-tensioned cables which tied balancing pairs of girders together. After study of the models and discussions with the Australian Welding Research Association the fabricator elected to cast this joint. Following ultrasonic and dye penetrant inspection the faces of the casting were machined prior to welding to the boom and post members (see Fig. 7).

Tower construction

As construction of the concrete podium levels proceeded, it became evident that completing the concrete plantroom structure at the level immediately under the first steel-framed tower floor would be an awkward and time-consuming process. To avoid any risk of delay to the tower erection programme, temporary columns were used to bypass the complex areas. Six temporary steel supports were erected up to level 9 and braced to the core structure. This enabled the six steel façade columns which commenced at level 9 to be installed. The temporary structure was designed to carry the construction load from a limited number of tower floors, sufficient only to enable the concrete structure below to catch up. Since the temporary members were located within the volume of the concrete columns they were progressively encased and the bracing removed.

Slipforming of the tower core and corner columns was programmed to proceed floor to floor on a weekly cycle. A trailing scaffold gave access to the exposed plates set in the walls and enabled the seating cleats to be surveyed and field-welded into their correct location (see Fig. 6).

Erection of the steel superstructure, which, because of the storey-height vierendeels required a two-level cycle, was preplanned and established on a beam-by-beam basis. While there was in practice a variety of sequences which could have been acceptably followed, this approach ensured that a successful erection method was adopted from the beginning.

Erection of the vierendeel girders which weighed up to 9 tonnes each was found to be straightforward. After positioning on their seating they were readily restrained by anchor bolts.

Connection of the steel to the slipformed concrete was by means of two post-tensioned stressing cables, each comprising 11 strands of 12.5 mm diameter. This necessitated pouring an infill concrete joint to ensure a matching fit. To avoid a delay to the erection sequence, the high tensile steel anchor bolts, as temporary restraint, were sized to carry, in addition to the dead weight of the girders, the relevant beam, decking and construction loading. This enabled steel erection and preparation for pouring the concrete floor slab to continue to two floor levels while the jointing concrete gained strength.

In retrospect the only difficulty which arose was an unusual one. The degree of tolerance which had been built in to the design enabled the structure to fit together in a very wide range of circumstances. This could give a false sense of security to the erection team, in that irregularities which would normally have become immediately evident, were not necessarily disclosed. The survey controls to ensure that the structure was correctly aligned were consequently more extensive than might normally have been anticipated.

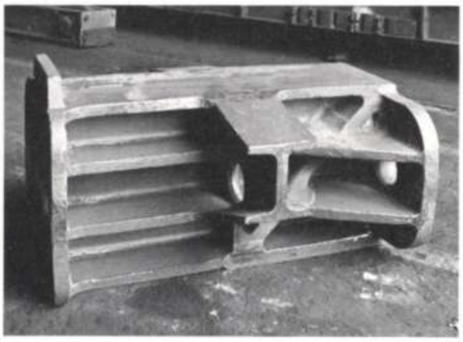


Fig. 7
Casting for corner section of vierendeel girder (Photo: Max Dupain)

Banking chamber roof

The banking chamber roof covers a large irregular-shaped area. Four plate girders, the largest spanning 25 m, support transverse trusses which cantilever varying distances up to 7 m. A perimeter truss connecting the ends of the cantilevers served to stabilize concrete cladding units as well as to even up relative deflections. A concrete roof slab was poured on decking on the top chord of the trusses. Ferro-cement panels and lighting fixtures are suspended below. A large services void is available within the depth of the trusses. The effect of the irregular shape was compounded by a requirement for staged occupation of the banking chamber. This led to one end of the principal girder being supported on a temporary column for some 12 months.

When it became possible to erect the permanent column, not only was the banking chamber in use but also a 13 m high façade which incorporated 8 tonnes of glass had been installed! This glass wall was attached to, and carried by, triangular steel mullions which were hung from the perimeter truss connecting the cantilevers. The transfer operation had to be carefully controlled.

The deflections between adjacent window mullions were precisely calculated and using a flat jack, movements were monitored and compared with predicted values.

The Wynyard Arcade

The alignment of the Wynyard Arcade takes it below Margaret Street and directly under Carrington to give a pedestrian network connecting Wynyard Concourse through the CBA Centre arcades to George Street. Margaret Street carries significant Harbour Bridge traffic while Carrington Street is a major marshalling area for Urban Transport Authority buses. It was therefore important that there should be minimal disturbance at ground level and vital that traffic flows should be maintained at all times. Additionally it was not acceptable to excavate from the ground surface in Wynyard Park except in one location where stair access to the arcade was required. Approximately

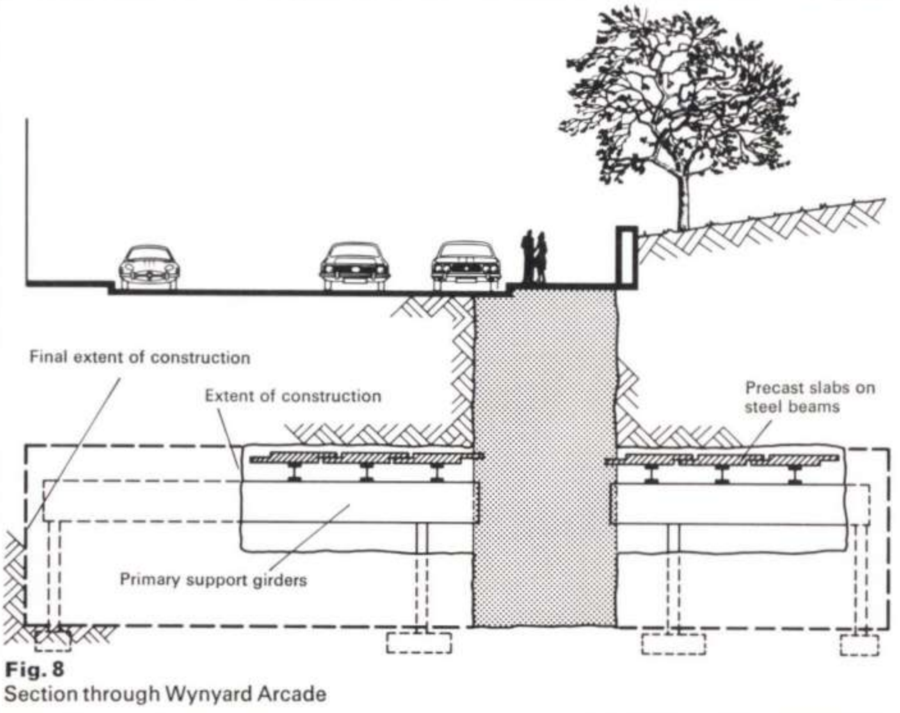


Fig. 8
Section through Wynyard Arcade

Fig. 9
CBA Centre tower (Photo : Max Dupain)

24 m in overall width, the arcade includes retail shops along its length of 55 m and it varies in depth between 2.5 and 5 m below street surface level.

The construction sequence had to be carefully planned to meet authorities' requirements and this to a large extent dictated the construction methods.

Only a small section of the arcade could be constructed as an open cut. For the greater part, mining techniques were adopted. A 4 m wide trench was excavated on the axis of the arcade. This was sufficiently narrow to be bridged over where necessary to maintain traffic flows. From this trench, transverse shafts were excavated. Heavy plate girders supported on steel columns were erected in these tunnels and carefully positioned for line and level. Secondary steel beams with 1 m wide concrete slabs precast compositely on the top flange spanned between the main girders. They were installed by excavating the rock pillars between the tunnels progressively away from the trench. The minimum excavation necessary to erect a single beam was completed; the beam was then installed and the rock overburden packed off from the top surface of the concrete. This sequence was repeated until the full arcade width had been excavated (see Fig. 8).

The design of the CBA Centre commenced when a decline in building activity in Sydney was becoming apparent following boom conditions in the first half of the 1970s. While the construction undoubtedly benefited from the comparative lack of industrial unrest it is also clear that the speed of construction was significantly enhanced by the use of structural steel in the tower.

Construction commenced on site in January 1976. Phased completion enabled the first section of the podium to be occupied early in 1978 while the final stage was handed over in October 1979.

The client's decision to proceed at a time of economic uncertainty has been rewarded with the lettable office space coming on to the market at a time when an upsurge in business activity has led to an improved demand and the building has been fully leased. In June 1981 the building was sold to the State Superannuation Board at an Australian record price for a building of $122 m.

Credits

Client:
Commercial Bank of Australia Ltd.

Architect:
Peddle Thorp & Walker

Structural engineer:
Ove Arup & Partners

Main contractor:
Concrete Constructions (NSW) Pty. Ltd.

Printed by Thompson & Tompkins Mitcham England 36658